Design and Analysis of Chemical Coagulation Systems to Enhance the Performance of Waste Stabilization Lagoons

by

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Submitted to the Department of Civil and Environmental Engineering on
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Master of Engineering in Civil and Environmental Engineering

Abstract

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Chapter 1 - Introduction

In October 1998, a conceptual project began at the Massachusetts Institute of Technology (MIT) for feasible wastewater lagoon treatment in the state of Sao Paulo, Brazil. The chosen treatment method was Chemically Enhanced Primary Treatment. This is, as the name states, the enhancement of conventional primary treatment via chemical addition. But, in the undertaken project, it was not the sole treatment, it was pre-treatment and in-pond treatment to enhance the performance of waste stabilization lagoons and the direct application to enhance lagoon systems. There are different means for chemically enhancing treatment (discussed in Chapter 7). Thus, for the purposes of this thesis, the acronym CEPT is used as the general term for enhancing conventional primary sedimentation basins, whether or not followed by lagoons. And for chemically enhancing waste stabilization lagoons (by dosing in the lagoon), it is called "in-pond CEPT."

It soon came to be that there was an actual wastewater facility soon to be built in Brazil. Through contacts with Brazil, the MIT group was given permission to visit the current facility, view the existing facility as well as study the design of the proposed facility to be built in its place, and given the opportunity to create their own alternative design(s). The design(s) by the MIT group would be considered as a possible replacement of the proposed design for the upgrading of the current (severely overloaded) facility.

In January 1999, a group went to Tatui, Sao Paulo, Brazil to conduct a field study of Tatui's wastewater treatment facility (CEAGESP). The group consisted of Dr. Donald Harleman (Ford Professor Emeritus at MIT), Susan Murcott (Research Affiliate at MIT), Christian Cabral (MIT graduate student), Frederic Chagnon (MIT graduate student), and Domagoj J.

Gotovac (MIT graduate student). Field tests and jar tests were undertaken to help the group better understand the efficiency of the present treatment system in Tatui, and to accomplish the common goals of jar tests, which include the determination of the optimum chemical coagulant dosage, optimum polymer dosage, optimum coagulant/polymer combination, and the necessary mixing and settling time for the optimum dosage/combination. The goals of the project were to design a facility that would treat the wastewater to the effluent standards set forth by SABESP (Sao Paulo's environmental agency) and to do so with the limitation that the designed facility must occupy an area no greater than that currently occupied by CEAGESP (the name of the facility currently in place in Tatui).

Each design alternative was done to achieve the effluent limit of 60 mg/L BOD_5 (5-day biochemical oxygen demand), which was the only specified effluent parameter for the design of the treatment systems. The average influent characteristics, provided by SABESP specifications for design, include the following: inflow rate (Q) = 161 L/s; influent $[BOD_5]$ = 276 mg/L; and influent [TSS[1]] = 200 mg/L.

The subsequent chapters will cover the relevant aspects involved in designing a wastewater facility utilizing chemical coagulation as a means of enhancement. Chapter 2 will analyze the current design and the three design alternatives. Chapter 3 will explain the theory of chemical coagulation and flocculation in terms of the surface chemistry of colloidal particles, since coagulation and flocculation are the "enhancing" portion of any form of CEPT. Chapter 4 will introduce CEPT and give some data and examples of its usage. Chapter 5 will analyze

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[1] TSS: Total Suspended Solids.

the jar test data, analyze other tests conducted in the field visit (such as the efficiency of the present lagoon system), and will analyze the different chemicals and polymers used in the jar tests. Chapter 6 will discuss various handling methods for sludge and discuss the chosen sludge-handling methods for each design alternative. Chapter 7 will discuss design considerations and the design of the two alternatives designed by the MIT group. The last chapter, Chapter 8, will conclude with the choice of the optimum design alternative.

Chapter 2 - Current Design and Design Alternatives

Introduction

This chapter will briefly describe the current design and each design alternative. It should be noted that each alternative will use the combined bar screen-grit chamber unit designed by SABESP for Alternative 1.

Design Alternatives

Current Design

The current treatment system of CEAGESP consists of0 an anaerobic lagoon followed by a facultative lagoon. The system is severely overloaded, which is why it will be upgraded. See Figures 2-1 and 2-2.

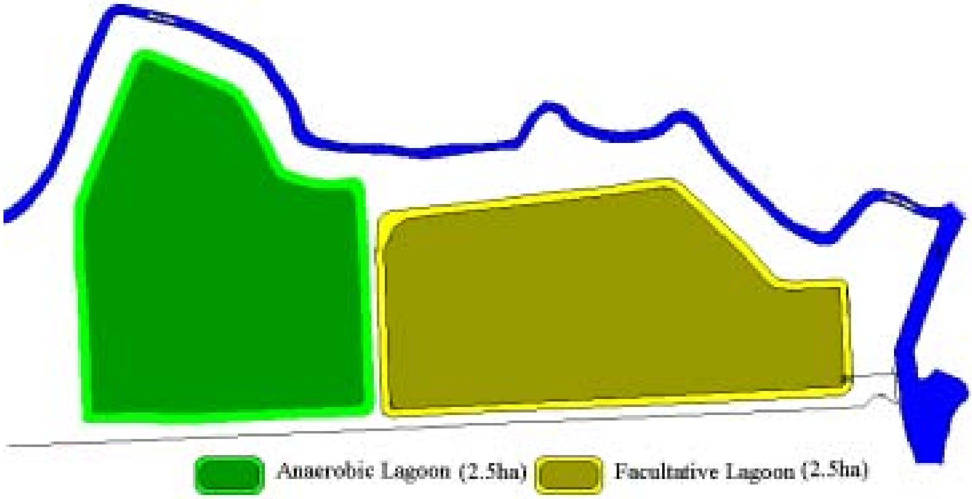

Figure 2-1: CEAGESP Layout

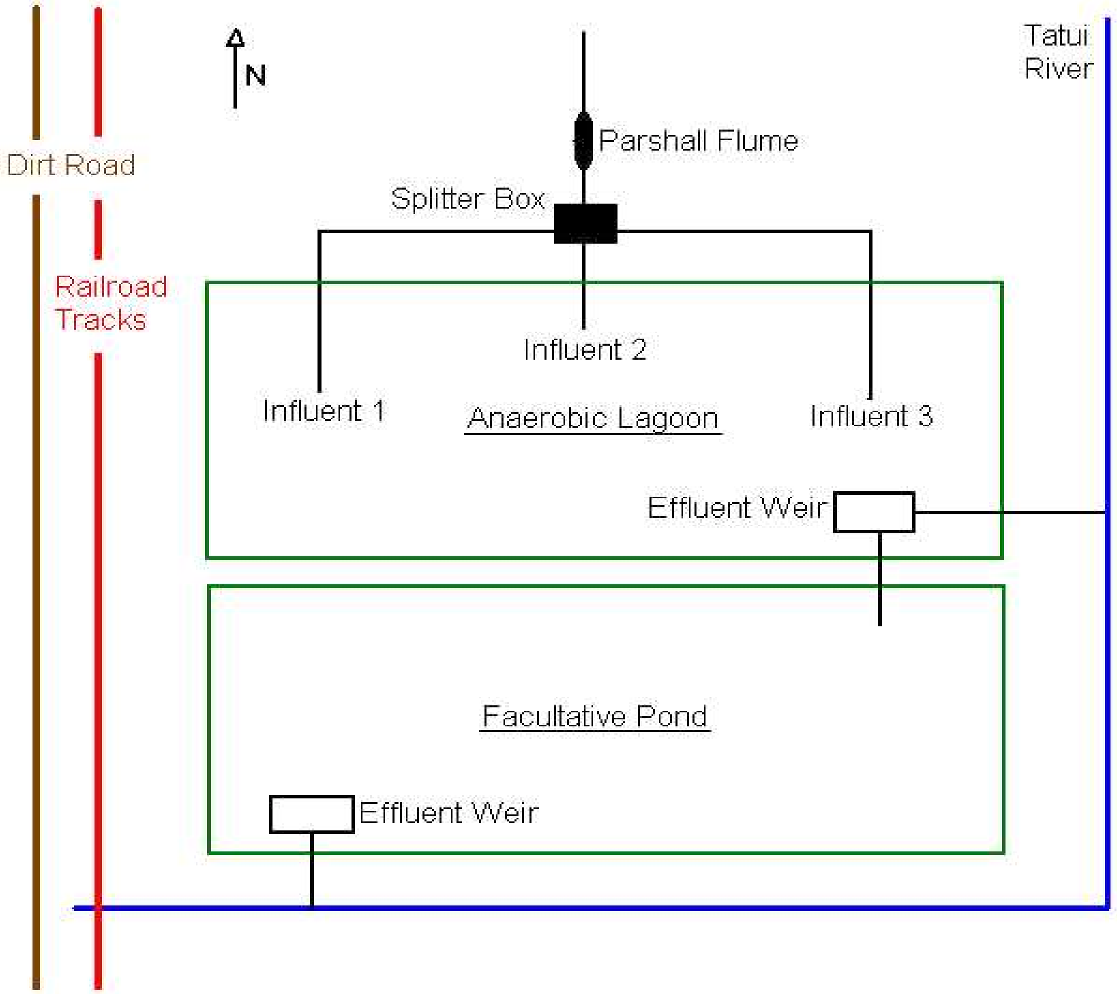

Figure 2-2: CEAGESP Schematic

As can be seen in the schematic of the CEAGESP facility in Figure 2-2, part of the anaerobic effluent is directly discharged into the river. After the anaerobic lagoon, the other portion goes into the facultative lagoon. As stated, the system is severely overloaded and is thus operating well below design expectations. From the results of the field sampling and testing at CEAGESP, it was determined that the anaerobic lagoon had a COD removal efficiency of 35%, whereas a properly operated anaerobic lagoon should remove 50-85% of the BOD_5 (Metcalf & Eddy, 1991). The facultative lagoon had a COD removal efficiency of

26%, whereas a properly operated facultative lagoon should remove 80-95% of the BOD_5 (Metcalf & Eddy, 1991). It is often found that BOD_5 removal does not equal COD removal, but they are related, and removal efficiencies are close. Thus, although it can not be stated, for example, that the facultative lagoon is only removing 26% of the expected 80-95% of the BOD_5, it is certain that the system is not performing up to par. Nevertheless, the COD measurements are a useful indicator of its current level of efficiency, or lack there of.

Alternative 1

The first alternative is SABESP's design. It is an aerated lagoon system which consists of aerated lagoons followed by settling lagoons (the lagoons were often referred to as "tanks" by SABESP officials, thus the labeling in Figure 2-3). The sludge is dried in the sludge drying beds upon conveyance to them by a pump barge. [See Figure 2-3.]

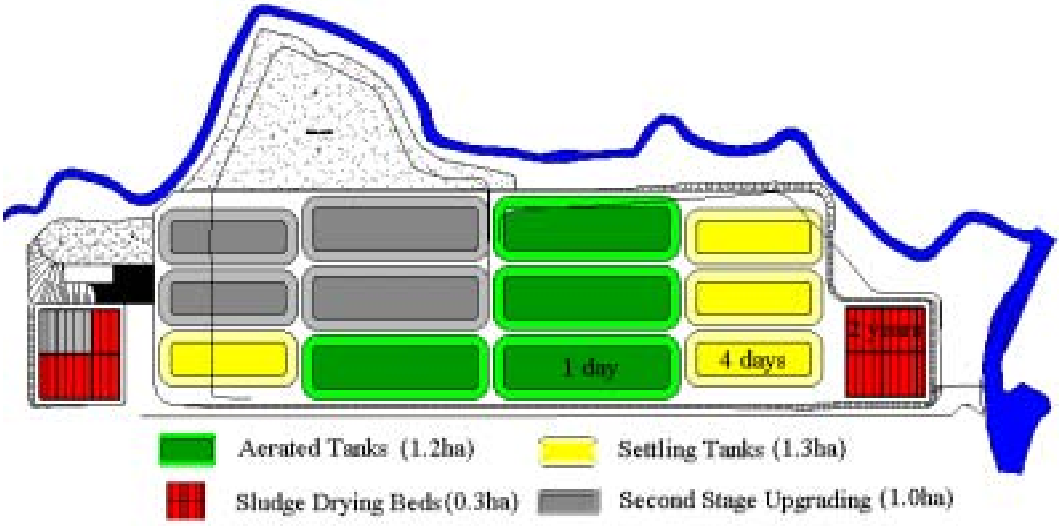

Figure 2-3: Layout of Alternative 1 (SABESP's Design)

Figure 2-3 shows more than four aerated lagoons and four settling lagoons. This is because the SABESP design calls for building four aerated lagoons and four settling lagoons at first, then upgrading the facility by adding two more settling lagoons in the future. This expansion also entails building more sludge drying beds and purchasing more surface aerators.

The current upgrade consists of four aerated lagoons whose total surface area is approximately 12,000 m^2, with a depth of 3.5 m. Thus, the total volume is 42,000 m^3, which yields a hydraulic retention time of 3 days. The settling lagoons have a total surface of 7000 m^2 and a depth of 3 m. Thus, the total volume is 21,000 m^3, yielding a hydraulic retention time of 1.5 days.

Alternative 1 Design: Aerated Lagoons

Alternative 1 is an aerated lagoon system followed by settling lagoons. The system consists of four aerated lagoons equipped with five aerators each rated at 15 hp. Four settling lagoons follow these aerated lagoons. The settled sludge will remain in the lagoon for two years (under which it will digest and become stabilized) and will subsequently be pumped into the sludge drying beds. The design was by SABESP, and no analysis can be performed on the methods of design since the calculations are undisclosed. But, in order to determine the feasibility of using an aerated lagoon system, the following calculations were done to determine the necessary horsepower (the calculations are adapted from Metcalf & Eddy (1991)):

The design assumptions are as follows:

Q = 161.01 L/s = 3.68 MGD

Soluble Influent BOD_5 = 150 mg/L

Soluble Effluent BOD_5 = 20 mg/L

Influent Suspended Solids are not biologically degraded

Influent SS = 200 mg/L

Effluent Suspended Solids after settling = 60 mg/L

Kinetic Coefficients:

Y (maximum yield coefficient $= \dfrac{Mass_{NewCells}}{Mass_{SubstrateConsumed}}$)= 0.65

Ks ([substrate] at ½ of maximum growth rate) = 100 mg/L

k (maximum substrate utilization rate) = 6.0 d^{-1}

k_d (endogenous decay coefficient) = 0.07 d^{-1}

Total biological solids produced are equal to computed VSS ÷ 0.80

First-order soluble BOD_5 removal-rate constant k_{20} = 2.5 d^{-1} @ 20ºC

Summer Air Temperature = 30ºC (86ºF)

Winter Air Temperature = 10ºC (50ºF)

Wastewater Temperature = 15.6ºC (60ºF)

Temperature Coefficient: $\theta = 1.06$

Aeration Constants: $\alpha = 0.85$, $\beta = 1.0$

Elevation (of aerated lagoon system) = 2000 ft (610 m)

Lagoon Depth = 3.5 m (11.5 ft)

Oxygen Concentration to be maintained = 1.5 mg/L

Lagoon Surface Area = 12,400 m^2 = 133,486 ft^2

Design Mean Cell-Residence Time, θ_C = 3 d

1. Estimate summer and winter liquid temperatures:

Summer: $T_W = \dfrac{(133{,}486)(12x10^{-6})(86) + (3.68)(60)}{(133{,}486)(12x10^{-6}) + 3.68} = 67.9^\circ F$

Winter: $T_W = \dfrac{(133{,}486)(12x10^{-6})(50) + (3.68)(60)}{(133{,}486)(12x10^{-6}) + 3.68} = 57^\circ F$

2. Estimate the soluble Effluent BOD$_5$ measured at lagoon outlet during the summer:

$$S = \frac{K_S(1+\vartheta k_d)}{\vartheta(Yk - k_d) - 1} = \frac{100[1+(1)(0.07)]}{3[(0.65)(6) - 0.07] - 1} = 10.2 mg/L$$

3. Estimate the effluent BOD$_5$ with *k* adjusted for temperatures:

Summer: $\dfrac{S}{S_0} = \dfrac{1}{1+k\vartheta} \Rightarrow \dfrac{S}{150} = \dfrac{1}{1+(2.71)(3)} \Rightarrow S = 16.43 mg/L$

Winter: $\dfrac{S}{150} = \dfrac{1}{1+(1.7)(3)} = 25.0 mg/L$

Ratio of $\dfrac{S_{winter}}{S_{summer}} = \dfrac{25.0}{16.43} = 1.52$

Applying the ratio to the soluble effluent BOD$_5$ computed in part 2 yields a value of about 15.5 mg/L.

4. Estimate the concentration of biological solids produced:

$$X = \frac{Y(S_0 - S)}{1 + k_d\vartheta} = \frac{0.65(150 - 10.2)}{1 + (.07)(3)} = 75.1 mg/L \text{ VSS}$$

5. Estimate the TSS in the lagoon effluent before settling:

$$SS = 200 mg/L + \frac{75.1 mg/L}{0.80} = 294 mg/L$$

6. Estimate the oxygen requirement:

lb O_2/d = $\dfrac{Q(S_0 - S)x8.34}{f} - 1.42P_x$

$$P_x = (75.1mg/L)(3.68MGD)[8.34lb/Mgal \cdot (mg/L)] = 2305lbO_2/d$$

Now, assume the conversion factor for BOD_5 to BOD_L is 0.68, determine the oxygen requirements.

lb O_2/d = $\dfrac{(3.68MGD)[(150 - 10.2)mg/L(8.34)]}{0.68} - 1.42(2305lb/d)$

$= 3037lb/d = 1379kg/d$

7. Compute the ratio of oxygen required to BOD_5 removed:

$$= \frac{3037lb/d}{[(150 - 10.2)mg/l(3.68MGD)(8.34)} = 0.71$$

8. Determine the surface aerator power requirements, knowing that the aerators used are rated at 2.86 lb O_2/hp·h

   i. Oxygen saturation at 21.2C = 8.87 mg/L
   ii. Corrected for altitude, = 8.34 mg/L
   iii. C_{s20} = 9.08

Thus,

$$= \left[\frac{\beta C_{walt} - C_L}{C_{S20}} 1.024^{T-20} \alpha\right]$$

Correction Factor : $= \dfrac{8.34 - 1.5}{9.08}(1.024^{21.2-20})0.85$

$= 0.67$

The field-transfer rate N is equal to

$N = N_0$ (0.67) = (2.86)(0.67) = 1.92 lb O_2/hp·h

The amount of O_2 transferred per day per unit is equal to 46.04 lb O_2/hp·d. The total power required to meet the oxygen requirements is

$$hp = \frac{3037 lbO_2 / d}{46.04 lbO_2 / hp \cdot d} = 66 hp$$

9. Check the energy requirements for mixing. Assume that for a completely mixed-flow regime, the power requirement in 0.6 hp/1000 ft^3.

(a) Lagoon Volume = 1,532,812 ft^3

(b) Power required = (0.6)(1533) = 920 hp (685 kW)

Thus, if surface aerators rated at 15 hp were to be used, 62 of them would need to be used to properly mix this specified volume of wastewater (3.68 MGD) and lagoon volume (1,532,812 ft^3); SABESP's design (Alternative 1) calls for the use of 20 of these same surface aerators.

Alternative 2

Alternative 2 is the first of two alternative design proposals by the MIT group. The treatment system consists of three chemically enhanced sedimentation basins followed by an anaerobic lagoon, followed by a facultative lagoon. The sludge from the chemically enhanced sedimentation basins will be pumped to a filter press and subsequently composted (windrow composting). See Figure 2-4 for its layout.

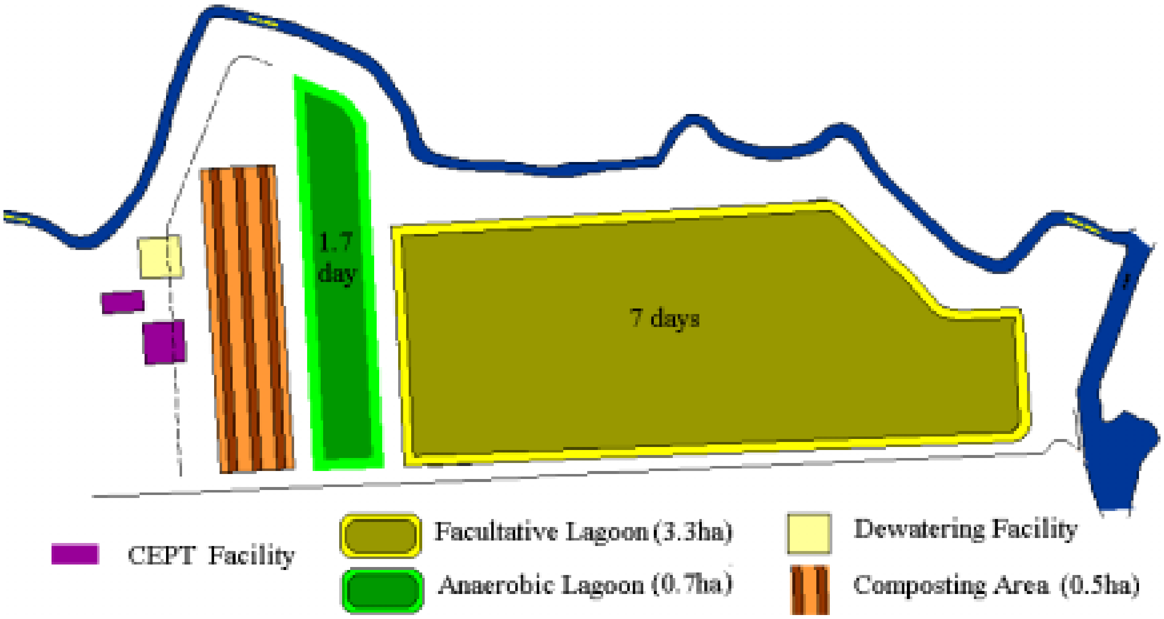

Figure 2-4: Alternative 2 Layout

Alternative 3

The third alternative is also a design of the MIT project. It is an in-pond CEPT facility. The wastewater first enters a CEPT lagoon (called a “CEPT settling lagoon” in Figure 2-5). Then the wastewater proceeds into an anaerobic lagoon, and then into a facultative lagoon. The sludge in the in-pond CEPT lagoon will be pumped out by a pumping barge after a two-year residence time and will be dried in sludge drying beds. See Figure 2-5 for its layout.

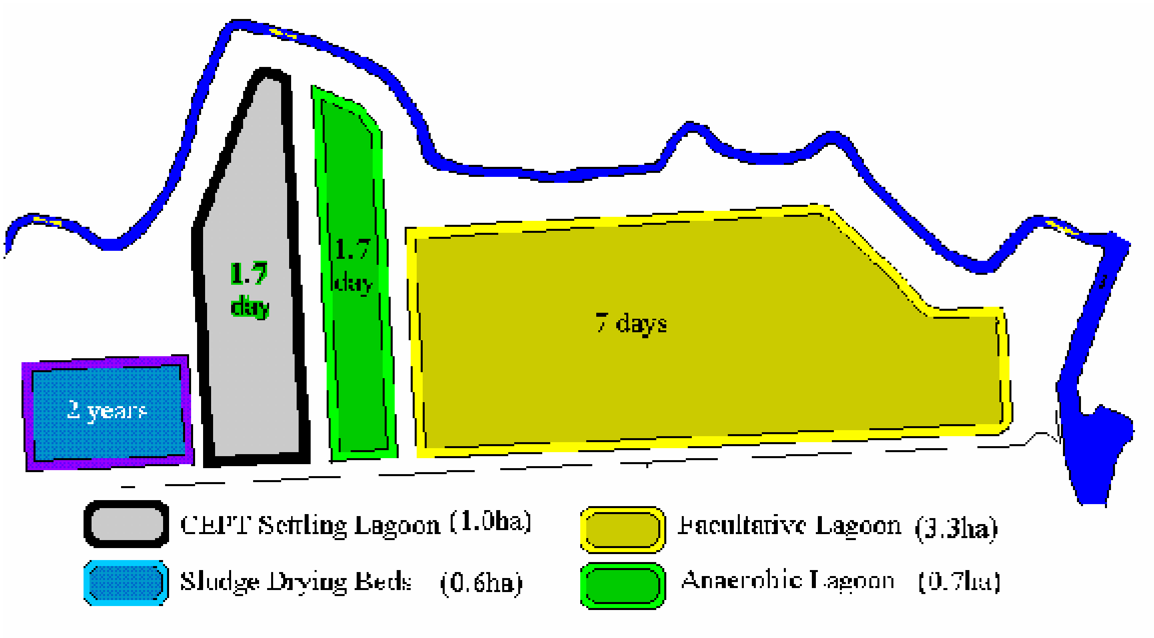

Figure 2-5: Alternative 3 Layout

Chapter 3 - Coagulation and Flocculation: Colloidal Surface Chemistry

Introduction

This purpose of this chapter is to describe the basic surface chemistry of colloids, the processes of coagulation and flocculation, and their relation to wastewater treatment. Coagulation and flocculation are important processes utilized in many applications, particularly water treatment, domestic wastewater treatment, and industrial wastewater treatment. Significant in the processes of coagulation and flocculation are the removal of colloidal particles, for which their surface chemistry needs to be understood. This knowledge is also important in many other applications, such as adhesion, precipitation, detergency, food processing, sugar refining and heterogeneous catalysis, just to name a few (Shaw, 1992).

Overview of Coagulation, Flocculation, and Colloidal Surface Chemistry

Surface chemistry can be defined as the study of the interfaces between two bulk phases in contact. A colloidal system is defined as a system in which particles, in a finely divided state, are dispersed in a continuous medium. The particles are called the "dispersed phase", and the medium in which the dispersed phase exists is called the "dispersing phase" (Benefield *et al.*, 1982). A colloidal dispersion has no net electrical charge.

Colloids are very small particles and/or large molecules, which can be considered to be in the range of 10^{-6}m to 10^{-9}m range (this range is not to be taken as exact, as many authors abide by a different particle range for colloids). For a size classification see Figure 3-1. Colloids can be solids, liquids or gases. They include aerosols, agrochemicals, cement fabrics, foodstuffs, paper, pharmaceuticals, plastics, rubber, clays, and emulsions (Shaw, 1992). See Table 3-1 for types of colloidal dispersions. Colloids are very fine solids which are not thermodynamically stable (total surface energy is greater in the dispersed state that in the aggregated state), which is the reason they are considered virtually "non-settleable," that is, without the aid of coagulants/flocculants (Hering *et al.*, 1993). Coagulation and flocculation reduce the total free energy of a system of colloidal particles, which allows aggregation to occur.

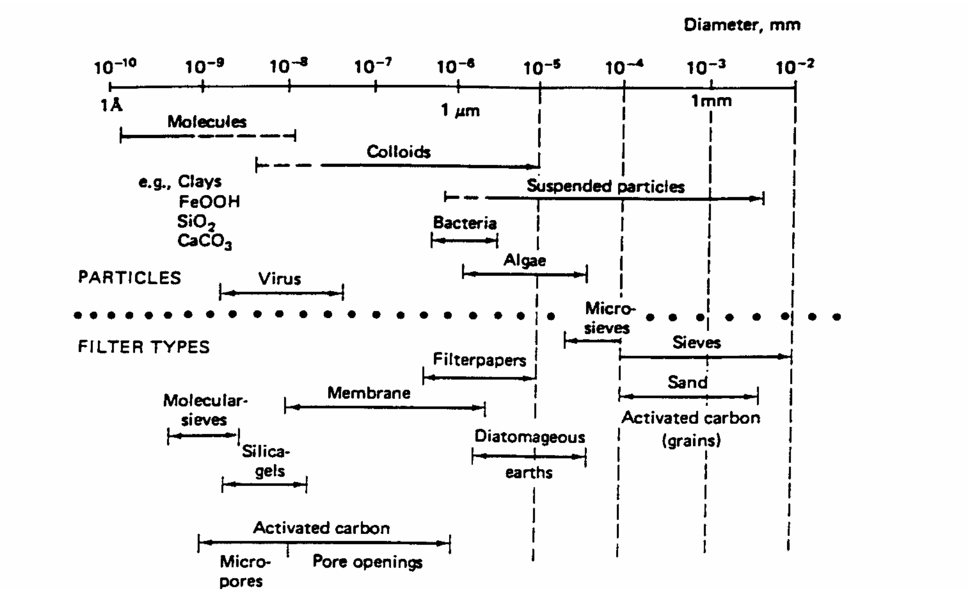

Figure 3-1: Particle Size Classification[2]

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[2] Source: Benefield *et al.*, 1982

Table 3-1: Colloidal Dispersions[3]

Dispersed Phase	*Dispersion Medium*	Name	Examples
Liquid	Gas	Liquid aerosol	Fog, liquid sprays
Solid	Gas	Solid Aerosol	Smoke, dust
Gas	Liquid	Foam	Foam on soap solutions, fire extinguisher foam
Liquid	Liquid	Emulsion	Milk, mayonnaise
Solid	Liquid	Sol. Colloidal Suspension : Paste (high solid concentration)	Au Sol, AgI sol: toothpaste
Gas	Solid	Solid foam	Expanded polystyrene
Liquid	Solid	Solid emulsion	Opal, pearl
Solid	Solid	Solid suspension	Pigmented plastics

Coagulation occurs as particles are destabilized, which can be achieved by the addition of metal salts. Colloids do not aggregate on their own, therefore, coagulation is necessary to destabilize these particles to form aggregates. Coagulation follows three steps: the formation of the coagulant species upon entry into the liquid, destabilization of the particles, and interparticle collisions (Furuya *et al.*, 1998). The intermediate chemical species that form as a result of the rapid reactions of precipitation and hydrolysis are essential for particle destabilization (Furuya *et al.*, 1998).

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[3] Source: Benefield *et al.*, 1982

The types of metal salts that can be used include the following: Aluminum sulfate (alum), $Al_2(SO_4)_3 \cdot 14H_2O$, or $Al_2(SO_4)_3 \cdot 18H_2O$; Ferrous Chloride, $FeCl_2$; Ferric Chloride $FeCl_3$; Ferric Sulfate, $Fe_2(SO_4)_3$; Ferrous Sulfate, $FeSO_4 \cdot 7H_2O$; and Lime[4], $Ca(OH)_2$ (Reynolds *et al.*, 1996). Seawater is also occasionally used as a coagulant by coastal cities utilizing Chemically Enhanced Primary Treatment due to its content of metal salts. Rapid mixing is associated with coagulation because colloids coagulate at a rate dependant on the frequency of colloidal particle encounters; it is also dependant on the probability that their thermal energy is sufficient to overcome the repulsive potential energy barrier as these encounters take place (Shaw, 1992).

Precipitation is a part of coagulation. It is the conversion of a soluble substance into a solid (called a precipitate). This is the key component of sweep flocculation. When metal salts are added to a water sample, they rapidly form metal hydroxides[5], such as $Al(OH)_3$ and $Fe(OH)_3$. These are precipitants which, as they settle rapidly, carry along with them colloidal particles. The process is called precipitate/sweep coagulation/flocculation, or, enmeshment (Benschoten *et al.*, 1990). The precipitates themselves are called sweep floc, which entrap other particles and foreign ions into the precipitate itself, that is, jammed into the lattice.

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[4] Traditionally lime has been exclusively used, but it is not recommended, as it creates large amounts of sludge.

[5] The general expression for hydroxo-metallic complexes is Meq(OH)pz+ (Reynolds *et al.*, 1996). Aluminum salts form some of the resulting polymers: $Al_6(OH)_{15}^{+3}$, $Al_7(OH)_{17}^{+4}$, $Al_8(OH)_{20}^{+4}$, and $Al_{13}(OH)_{34}^{+5}$; for an iron salt, some of the resulting polymers are $Fe_2(OH)_2^{+4}$ and $Fe_2(OH)_4^{+5}$. The adsorption of (highly-charged) hydroxo-metallic complexes by colloids is responsible for the reduction of the zeta potential.

In the process of flocculation, a transport process (via gentle mixing), polymers of high molecular weight are utilized to create bridges between particles. The process of flocculation includes a binding mechanism that creates floc which are stable, and therefore settle (Dobias, 1993). It is a process in which the aggregated particles lose their kinetic independence. Flocculation can be seen as the process where synthetic organic polyelectrolytes (anionic, non-ionic, or cationic polymers) bind the formed precipitates from coagulation by their long-chained structure into larger particles. This, according to Stokes Law, will drastically increase their settling rate, as it is proportional to the square of the diameter.

Settling is modeled to follow Stokes Law:

$$V_c = \frac{g(\rho_s - \rho)d^2}{18\mu}. \tag{3-1}$$

Where:

V_c = Terminal velocity of particle (m/s)
ρ_s = Density of particle (kg/m^3)
g = Acceleration due to gravity (m/s^2)
μ = Dynamic viscosity ($N \cdot s/m^2$)
d = Particle diameter (m)
ρ = Density of fluid (kg/m^3)

Coalescence

Coalescence, as defined by Sonntag *et al.* (1972), is the destruction of the interlayers leading to particle fusion in foams and emulsions[6] or generation of direct contacts between solid

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[6] A system consisting of a liquid dispersed in an immiscible liquid. Immiscible liquids are ones which do not readily mix (such as oil and water). {Source: www.eb.com}

particles; it is all of the processes leading to direct contact of the particles. The interlayers referred to are the thin films[7] of the dispersing medium or adsorption layers of surface-active substances (surfactants). Random fluctuations, either thermal or mechanical, may cause the particles to leave the equilibrium state (when the attractive forces equal the repulsion forces) and approach one another to a distance which is even smaller. This type of instantaneous disturbance of the equilibrium layer leads to coalescence if the change in the repulsive forces is less than that of the attraction forces, i.e., if

$$\frac{d\Pi_D}{dd} > \frac{d\Pi_{el}}{dd} .$$

Where Π_D is the dispersion force per unit area and Π_{el} is the electrostatic force per unit area. The liquid film will be spontaneously ruptured if any change in the thickness of the equilibrium layer occurs.

The breakup of coalescence-stable layers was obtained from the analysis of emulsions and foams, not sols. Therefore, it will not be included here. Refer to Appendix A-6 for this analysis.

Colloids: Stability

Colloid stability, at its simplest, is assuming that lyophobic sols are stabilized, entirely, by the interactions of the electric double-layer. Lyophobic (or hyophobic) means "liquid-

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[7] Refer to Sonntag *et al.* (1972) for a description of the process of coalescence and its relationship to the layer of thin film.

hating"; sol is used to distinguish between colloidal dispersions and suspensions that are macroscopic (Shaw 1992), or, it can be seen to mean solids dispersed in liquids (Reynolds *et al.*, 1996), such as clay particles present in natural waters. The stability of colloids depends on many factors, including surface tension, ionic strength of electrolyte concentrations and crowding of polymer chains (Steric stabilization).

Colloidal stability can also be thought of as a particle's resistance to coagulation. Stable can be defined, practically, as the description of a dispersion for which the coagulation rate is very slow. This, in layman's terms, means that a destabilized particle can settle, and a stabilized particle won't settle, until destabilized.

Colloids in wastewater are stable due to their affinity to water (hydrophilic), and due to their surface charge (this is the case for hydrophobic particulates). Hydrophilic colloids, such as proteins, soaps and synthetic detergents, are very hard to remove from water and wastewater due to their affinity for water. A metallic salt dosage of an order of magnitude greater than that for the removal of hydrophobic colloids is often necessary to remove hydrophilic colloids. The surface charge of the hydrophobic particulates is usually negative because of the preferential adsorption of anions onto the surfaces of organic matter. Most organic and inorganic matter in water is hydrophobic, and depends on electrical charge for its stability in suspension. Since these particles are negatively charged, they adsorb positive ions onto their surface and repel each other since like charges repel.

It is a misconception that only colloids have a residual surface charge and other particles, smaller and/or larger, don't. All matter has this residual surface charge, to a certain degree, but the charge per unit volume of these small colloidal particles is what makes them stable (again, like charges repel). That is, colloids have a large specific surface area (surface area per unit volume). This large surface area leads to colloids' tendency to adsorb substances in the surrounding water.

Disperse systems for which the surface tension σ is zero or nearly zero may be considered as a special case of colloid stability, as stated by Sonntag *et al.* (1972). These dispersions are thermodynamically stable, that is, there is small interaction between the medium and the particles.

Coagulation structures can become stable for a variety of reasons, as listed above, and can become destabilized for a variety of reasons, as will be discussed. Often, these coagulation structures will restabilize, in a process called peptization; this is where electrolyte concentration is reduced (Sonntag *et al.*, 1972). These structures often become restabilized due to mechanical mixing which breaks the structure.

The stability of particles is also dependent on the ionic strength of electrolyte concentrations. Sonntag *et al.* (1972) state that at ionic strengths mostly below 10^{-3} mole/liter, binary collisions do not produce aggregates; this case is called flocculation-stable, whereas ionic strengths larger then 10^{-1} mole/liter are large enough for rapid coagulation (aggregation formation).

Steric stabilization occurs when particles are kept from flocculating. This stabilization is caused by the crowding of polymer chains within the overlap of particles. It involves adsorbed macromolecules "other than that of the double layer repulsion and the van der Waals forces" (Shaw, 1992). See Figure 3-2 for a visual description of the overlap of dispersed solid particles.

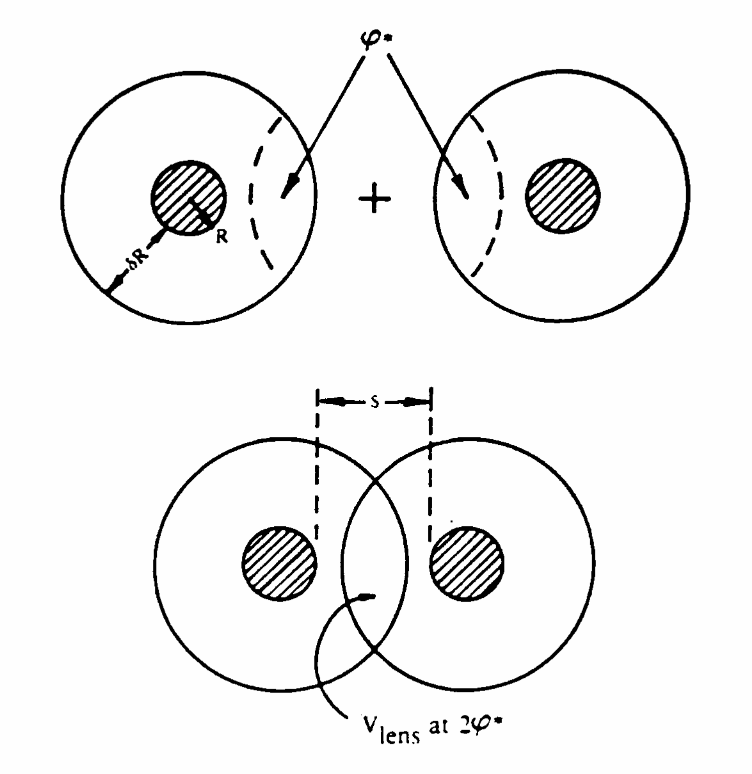

Figure 3-2: Overlap of Adsorbed Polymer Layers

"As the distance of separation between the core particles decreases in the flocculation step, the adsorbed layers begin to overlap as shown in the figure. Ultimately, it is the crowding of the polymer chains within this overlap column that produces any stabilizing effect observed. Consequently, this mechanism for protecting against flocculation is called steric stabilization." (Hiemenz, 1986)

Stability of Lyophobic Sols

The stability of lyophobic sols is limited. Stability of a system is lost when a coagulant is added and aggregation of colloidal particles ensues. The rate of coagulation will depend on the amount of collisions that take place. But, it is important to understand that not all collisions result in aggregation. The effectiveness of the collisions is affected by what is added into the system. Even small amounts of adsorbing substances added to the system can have an affect on this.

Fast coagulation is the term used to describe a state in which almost all, or all, of the collisions result in aggregation. The other side of the spectrum is slow coagulation in which not all of the collisions result in the formation of aggregates, that is, only a fraction (α) of them. In the case of fast coagulation, only the amount of collisions per unit time (frequency) is important in determining the rate of coagulation. In slow coagulation, the frequency of collisions and the surface properties of the particles are important (Sheludko, 1966). The effectiveness of collisions will depend on whether the van der Waals attraction forces are the predominant forces in the system, being greater than the repulsive forces which are a result of the electric double layer. Aggregation will decrease if repulsive forces dominate, leading to a higher stability.

The rate of coagulation is also dependant on the zeta (ζ) potential, a topic to be discussed later. When the ζ potential decreases, the rate of coagulation increases. This occurs at a low value of ζ. The point at which the fast coagulation occurs is know as the “critical potential” (Sheludko, 1966).

Heating of sols can be utilized as a process to stabilize the system. Heating the colloidal dispersion increases the particle motion and so the number of collisions. Electrolytes can then be added to reduce the electrostatic repulsion to create larger particles. This effect of the electrolytes is evident in a stream as it mixes with salt water. As the two waters mix, the salt water contains many polyvalent cations which will cause suspended clay particles in the river water to settle. This is what results in the formation of river deltas.

It should be noted that the addition of soluble lyophilic material is a method to enhance the stability of lyophobic sols. This material adsorbs onto the surface of the lyophobes and becomes a protective agent (Shaw, 1992).

The overall stability is a result of the attraction forces and repulsive forces. Figure 3-3 represents this as a function of particle separation. This figure demonstrates the exponential decrease of the repulsive energy between two particles as the separation increases; the van der Waals attractive force also decreases very rapidly with increasing intermolecular distances (Benefield *et al.*, 1982).

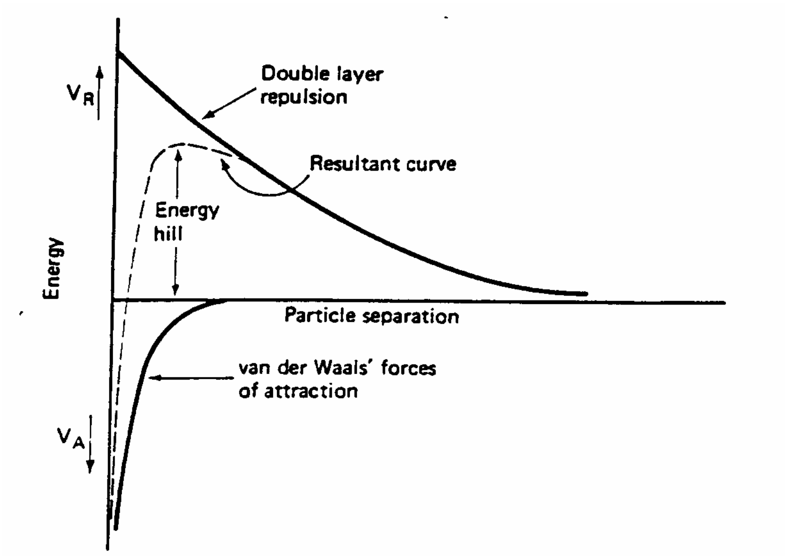

Figure 3-3: Repulsive and Attractive Energies as a Function of Particle Separation

"This curve indicates that repulsion forces predominate at certain distances of separation, but that if the particles can be brought close enough together, the van der Waals' attractive forces will predominate and the particles will coalesce. To come together, the particles must possess enough kinetic energy to overcome the so-called energy hill on the total energy side." (Benefield *et al.*, 1982)

Colloidal Particle Surface Charge

Colloidal particles carry an electrical charge. This is indicated by electrophoresis, which is when particles in a colloidal sol placed in an electric field move toward one of the electrodes (Benefield *et al.*, 1982). The electric charge is the primary reason for the stability of these particles and can be acquired in a number of ways. One way is by imperfections in the crystal structure. This occurs as a result of isomorphic replacements within the crystal lattice (Benefield *et al.*, 1982). This process is rare, but it is how clay particles in natural waters acquire their surface charge (Benefield *et al.*, 1982). Another way for a colloidal particle to acquire a surface charge is by adsorbing ions (usually anions) onto its surface. This is what happens to the organic matter in wastewater, as discussed above. These adsorbed ions are called peptizing ions (Benefield *et al.*, 1982). The surface charge can also be acquired through ion dissolution which is the result of uneven dissolution of oppositely charged ions onto the surface of the colloid. One more way is the ionization of surface sites. This is acquired via the ionization of surface functional groups (carboxyl, amino, etc.) (Benefield *et al.*, 1982).

Destabilization

There are three processes involved in the destabilization of particles. One is sweep coagulation, also known as enmeshment in a precipitate. Another is charge neutralization. This, due to the negative surface charge of particles in wastewater, is when the cationic metal salts dissociate in the water and compress the diffuse double layer around the particles. This compression enables van der Waals attraction forces to take over, as a result of the reduction of the zeta potential, and allow particles to aggregate/coalesce. The diffuse double layer, the zeta potential and van der Waals attraction forces will be described later in this chapter. The process of charge neutralization can only take place if the new compounds formed (in less than a second) from the addition of multivalent cationic metallic salts come into contact with the particles. This is achieved through rapid mixing. If destabilization is to occur, collisions must occur between the colloids, and the products of the metal hydrolysis and precipitation reactions must precede flocculation, which will occur due to rapid mixing (Amirtharajah *et al.*, 1986). The third process is interparticle bridging, which is accomplished by polymers. It is where the polymers gather and hold flocs that are already charge-neutralized, which is why this process is also associated with flocculation. A network is formed between the bridging of two particles that repel each other with other coagulated particles. The ionizable groups on the polymers bind with reactive sites or groups on the surfaces of the colloids. In this manner, several colloids may be bound to a single polymer molecule to form the bridging structure (Reynolds *et al.*, 1996). This network is called a floc. See Figure 3-4 for a schematic of reactions between colloidal particles and polyelectrolytes. See Figure 3-5 for a graphical presentation of the interparticulate forces acting on a colloidal particle [figures 3-3, 3-5 and 3-7 show similar representations of the concept, but it is easier to comprehend via inspection of figures; and the figures build off each other to better explain the interparticulate forces]. In Figure 3-5, the electrostatic zeta potential is the source of the repulsive forces and the van der Waals attractive forces are the source of the attractive force.

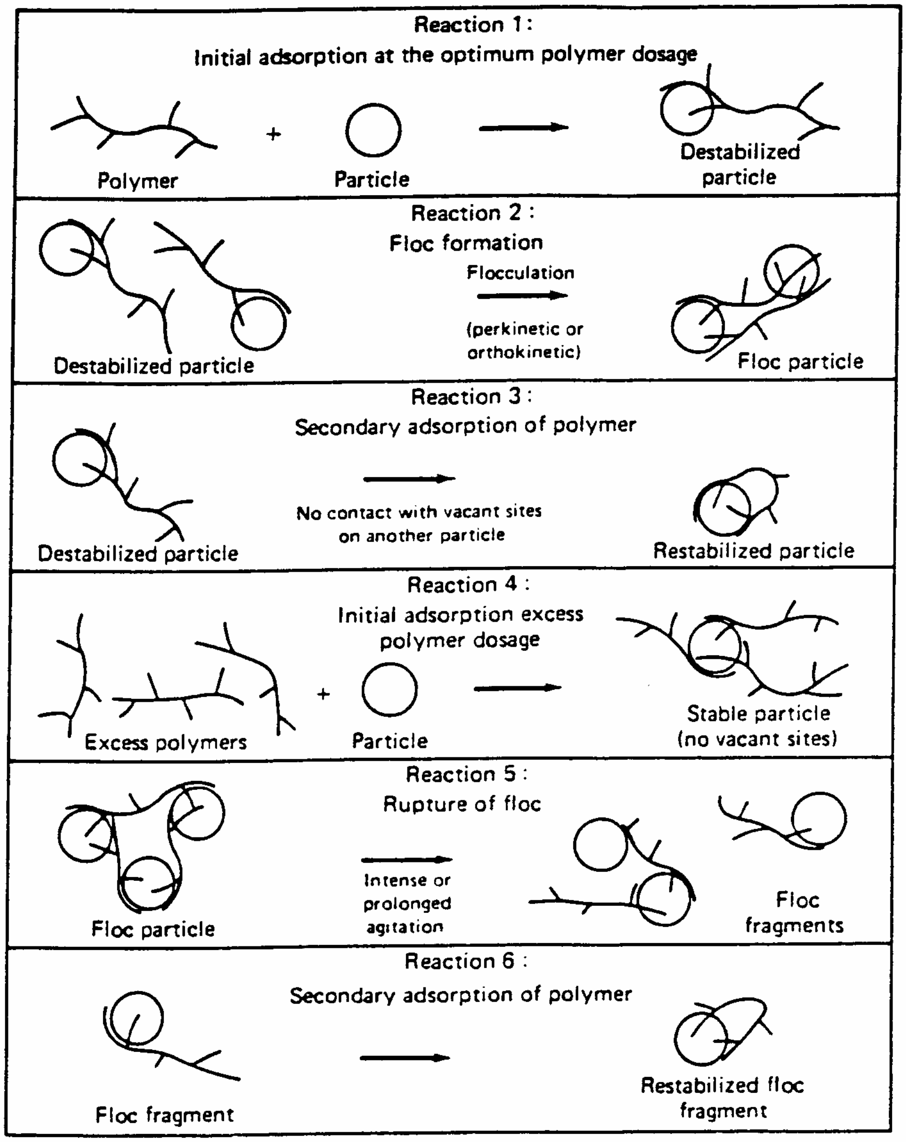

Figure 3-4: Colloidal Particle and Polyelectrolyte Schematic[8]

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[8] Source: Morrissey, 1990.

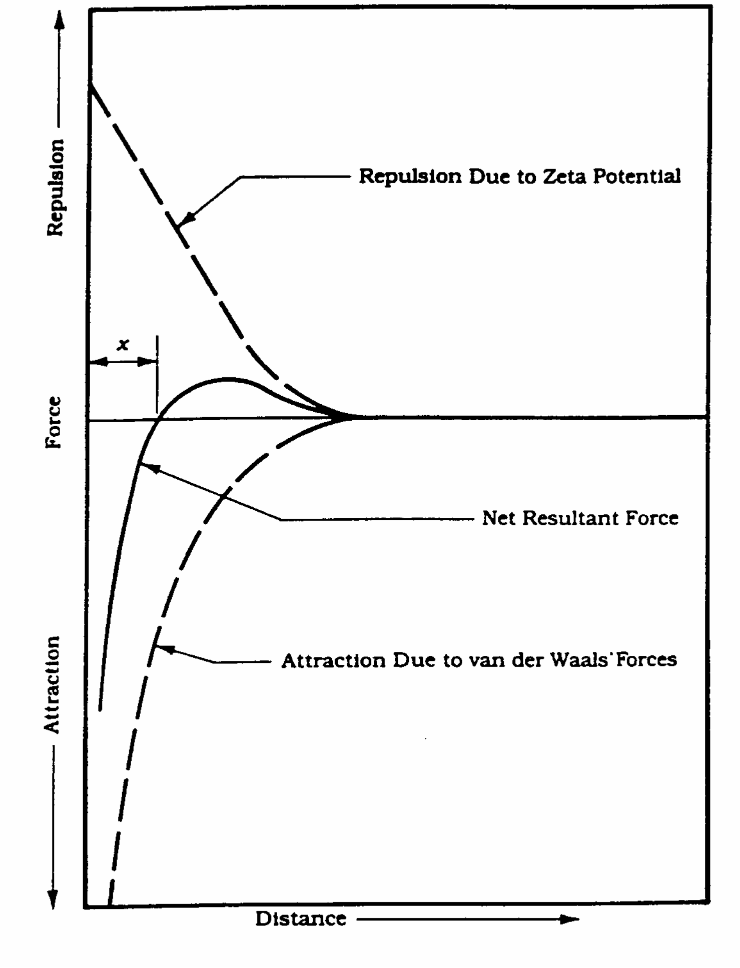

Figure 3-5: Colloidal Interparticle Forces v Distance[9]

The destabilization of colloids by adsorption of counter-ions is a process much different than that of the compression of the diffuse double layer. The difference between them is described by Benefield *et al.* (1982) to be mechanically different in three important ways. One is that double-layer compression ions need to be in a much larger concentration than those ions in adsorption. Secondly, destabilization by adsorption is dependent on the colloid concentration in the dispersion. It is stoicheometric, therefore, as the colloidal particle

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[9] Source: Reynolds *et al.*, 1996.

concentration increases so must the concentration of coagulant dosed. The importance in the increase in colloidal particles is not the amount of them, but the increase in the total surface area of the colloids. Thirdly, the system can be overdosed by excess coagulants which will restabilize the colloids due to charge reversal. This will change the negatively charged colloids into positively charged colloids, and thus repel each other once more.

Sensitization is another form of destabilization. It has been observed that floc stability decreases in the presence of the addition of macromolecular compounds. Often, the colloids begin to precipitate in the presence of these compounds. This phenomenon is important in the removal of suspended particulates in water. This occurs at low concentrations of the macromolecular compound (Sonntag *et al.,* 1972). The ensuing stabilizing effect of the macromolecular adsorption layers is often attributed to the weakening of the dispersion interaction.

See appendix A-4 for schematics relating coagulant dosage and colloid concentration, which is important to understand in sweep flocculation.

Particle Transport

Also known as flocculation, particle transport is needed to bring destabilized particles together, often by gentle mixing. The aggregation of coagulated particles will create larger particles, with (often) higher density and greater particle diameter, which will settle according to Stokes Law.

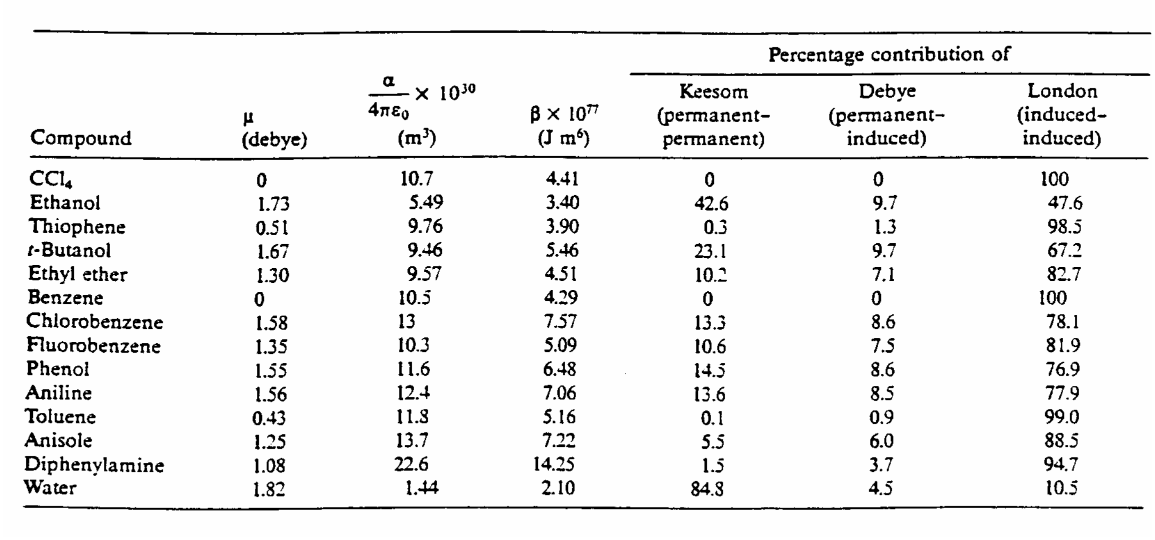
Three principal mechanisms can overcome the electrostatic energy barrier, collide, and coagulate. These collisions occur because of three mechanisms: Brownian motion (perikinetic flocculation), shear force (orthokinetic flocculation), and differential sedimentation (a special case of orthokinetic flocculation). Perikinetic flocculation is as the name implies, kinetic; it is due to motion, which is from the thermal energy of the fluid. Orthokinetic flocculation is caused by fluid motion, also kinetic, which is induced by mixing. Differential settling (sedimentation) is when particles settle rapidly and take with them smaller particles which are not settling, such as colloids, or particles with a lower settling velocity. This occurs because of exterior forces acting on the particles, which depends on the gravitational energy of particles (Hering *et al.*, 1993).

Attractive Forces

London Attractive Forces

The London attractive forces are the attractive forces which operate between non-polar molecules (see Appendix A-2 for an explanation of polarity). They are known as dispersion forces and are the result of charge fluctuation in a molecule associated with the motion of electrons (Sonntag *et al.*, 1972). These forces are extremely short-range and the force is inversely proportional to the intermolecular distance to the sixth power (Shaw, 1992). Dispersion forces are the only forces that act between nonpolar molecules (Sonntag *et al.*, 1972). London attractive forces are a component of the van der Waals attractive forces and are responsible for most of the van der Waals attraction. Table 3-2 demonstrates the contribution of London forces to the total van der Waals forces. This major contribution does not hold true for highly polar materials.

Table 3-2: Percentage of the DeBye, Keesom, and London Attractive Contributions to the van der Waals Attraction Between Various Molecules[10]



Compound	μ (debye)	$\frac{\alpha}{4\pi\varepsilon_0} \times 10^{30}$ (m^3)	$\beta \times 10^{77}$ (J m^6)	Percentage contribution of Keesom (permanent-permanent)	Debye (permanent-induced)	London (induced-induced)
CCl_4	0	10.7	4.41	0	0	100
Ethanol	1.73	5.49	3.40	42.6	9.7	47.6
Thiophene	0.51	9.76	3.90	0.3	1.3	98.5
t-Butanol	1.67	9.46	5.46	23.1	9.7	67.2
Ethyl ether	1.30	9.57	4.51	10.2	7.1	82.7
Benzene	0	10.5	4.29	0	0	100
Chlorobenzene	1.58	13	7.57	13.3	8.6	78.1
Fluorobenzene	1.35	10.3	5.09	10.6	7.5	81.9
Phenol	1.55	11.6	6.48	14.5	8.6	76.9
Aniline	1.56	12.4	7.06	13.6	8.5	77.9
Toluene	0.43	11.8	5.16	0.1	0.9	99.0
Anisole	1.25	13.7	7.22	5.5	6.0	88.5
Diphenylamine	1.08	22.6	14.25	1.5	3.7	94.7
Water	1.82	1.44	2.10	84.8	4.5	10.5

van der Waals Attractive Forces

According to Sonntag *et al.*, 1972, attractive forces between atoms and molecules other than chemical bond forces are known as van der Waals forces. The attractive force between particles can extend a considerable distance from its surface. The van der Waals attraction forces consist of three components. One is the London or dispersion force as discussed previously. Two others are the Keesom force (the dipole orientation force) and the DeBye force (induction force). These two forces require a permanent dipole moment for at least one of the two molecules involved in the interaction of the two particles (Hiemenz, 1986). See appendix A-5 for the Keesom, DeBye and London equations.

These forces manifest themselves in flocculation and are of great importance in disperse systems. The coagulated particles now (upon collision) have their electric double layer

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[10] Source: Hiemenz (1986).

depressed and can now flocculate. The van der Waals attraction forces play a major role in the agglomeration of colloidal particles.

The van der Waals attraction energy decreases as an inverse power of the distance between particles. They vary with the inverse of the fourth power of the interparticle distance, and with the electrostatic forces to the inverse of the second power (Hering *et al.*, 1993). Attraction forces take over when particles are very close together, which enables the subsequent coagulation of particles. But the attraction is weakened by the adsorbed layers of stabilizing agents (Shaw, 1992). Retardation[11] begins to enter the picture and effect the van der Waals forces at a distance of about 10nm (Hiemenz, 1986).

Repulsion

When particles are far apart, electrostatic repulsion creates an energy barrier that prevents coagulation and "stabilizes" the suspension. The like charge of particles is the reason repulsion occurs. If the diffuse double layers of two particles and thermodynamic equilibrium is maintained during the approach of two particles, the potential on the interface does not change while the charge does (charge decreases). This reduction in charge leads to repulsion between the particles. (Sonntag *et al.*, 1972)

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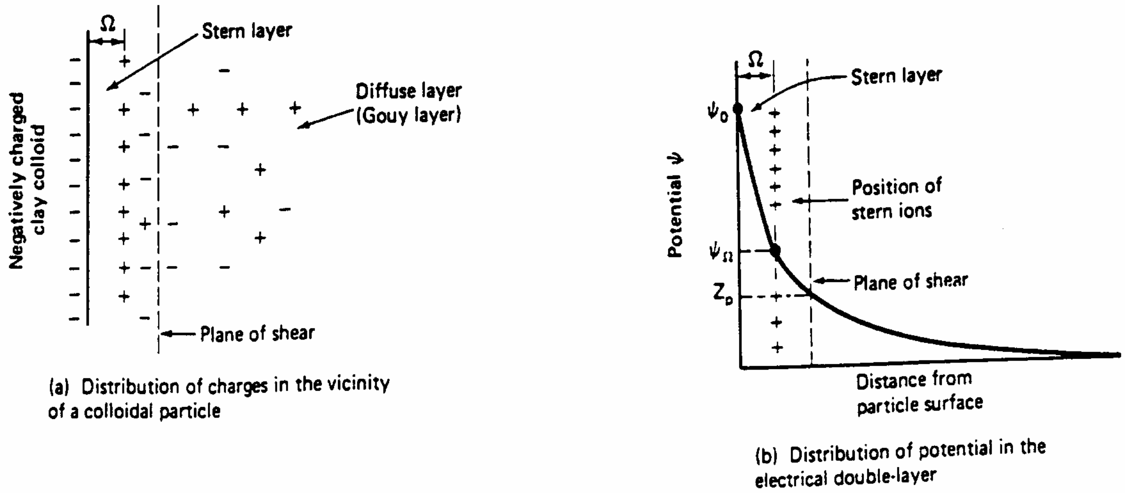
[11] This is beyond the scope of this thesis. Refer to Hiemenz (1986) for an analysis of retardation.

Diffuse Double Layer

This layer has many names, including the electric(al) double layer, the diffuse electrical double layer, the charged double layer, and others. The diffuse double layer is a result of the fact that colloids are charged and since a colloidal dispersion has no net electrical charge, the colloid is surrounded by an arrangement of ions of opposite charge to maintain electroneutrality. The surface charge consists of electrons or specifically adsorbed ions. The outer coating of the double layer is composed of the accumulated counter-ions which offset the charge of the particle. See Figure 3-6 for a graphical presentation and explanation of the diffuse double layer and its components.

It should be noted that the interaction of two particles will occur only if their diffuse double layers interpenetrate. The pH at which there is no double-layer interaction is called the iso-electric point (Shaw, 1992).

The compressing of the double layer does not involve a change in the total net charge. The thickness is reduced, thereby reducing the surface potential (associated by a decrease in the zeta potential) with increasing electrolyte concentration. This allows the van der Waals forces of attraction to be more dominant and allow aggregation to occur (Benefield *et al.*, 1982). This effect is depicted in Figure 3-7 (and Figure 3-3). As pointed out by Benefield *et al.* (1982), double-layer compression has two interesting aspects. One is that colloidal concentration in the dispersion does not have an impact on the amount of electrolyte necessary. And, no matter how much electrolyte is added, compression of the double layer cannot lead to charge reversal.



(a) Distribution of charges in the vicinity of a colloidal particle

(b) Distribution of potential in the electrical double-layer

Figure 3-6: Diffuse Double Layer: The Stern Model[12]

$$Z_p = \frac{4\pi q\delta}{D}$$

Where,

Z_p = Zeta Potential

q = Charge on the particle

δ = Thickness of the zone of the charge on the particle

And

Ω = Distance approximately equal to the hydrated radius of the ion

Ψ_Ω = Stern Potential

Ψ_O = Nernst Potential.

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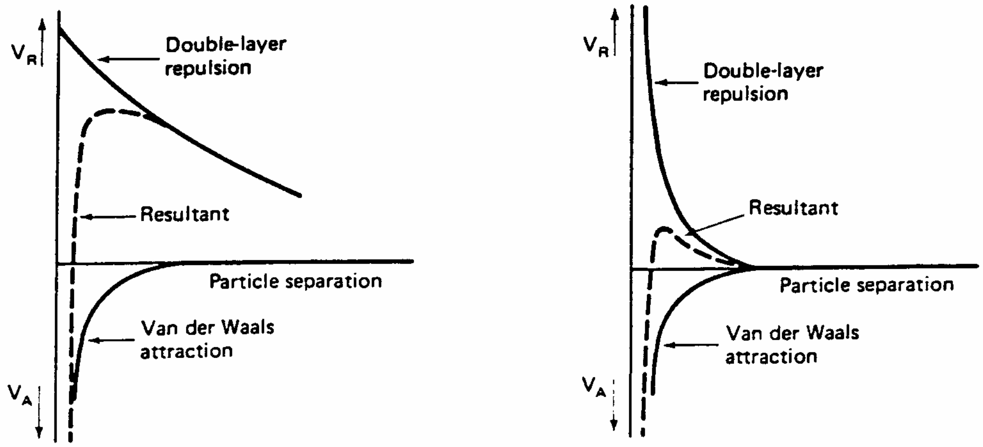
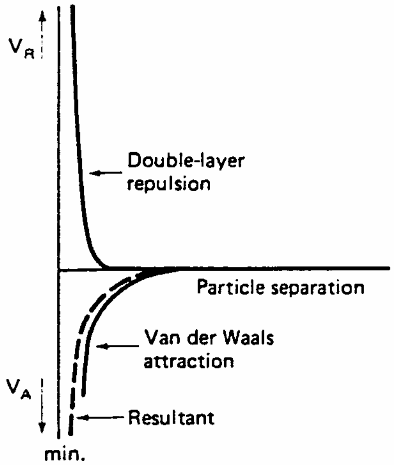
[12] Source: Benefield *et al.*, 1982.



(a) Low electrolyte concentration with normal double-layer thickness. System is stable and agglomeration is imperceptable.

(b) Intermediate electrolyte concentration causes some double-layer compression. Slow agglomeration can occur.



(c) High electrolyte concentration causes severe double-layer compression. Rapid agglomeration can occur.

Figure 3-7: Electrolyte Impact on the Diffuse Double Layer Compression[13]

The diffuse double layer can be split into three parts according to Sonntag *et al.* (1972) [see Figure 3-8]. The layer closest to the interface is formed by specifically adsorbed ions (the inner Helmholtz layer). Since the primary hydration layer is more cationic than it is anionic,

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[13] Source: Benefield *et al.*, 1982.

it has a preferential adsorption for anions. Beyond this layer, at a distance δ, the counter-ions (cations) compose the Outer Helmholtz or Stern layer. A potential drop occurs in this layer (linear drop, as Figure 3-8 shows). Beyond the Stern (outer Helmholtz) layer is the diffuse electrical double layer (Sonntag *et al.*, 1972).

If a stabilizing agent is added to the dispersion, is ionized, and carries a charge of the same sign as that on the particles, there will be an enhancement of the electric double layer repulsion. This is the case when an anionic surfactant is adsorbed on negatively charged particles. The adsorbed stabilizing agent will cause the Stern plane to move away from the particle surface; this will increase the range of electric double layer repulsion and, as such, enhance stability (Shaw, 1992).

Progressive particle agglomeration can occur as the repulsion force due to double layer suppression decreases[14]. This is accomplished by highly charged hydrolyzed metal ions. Figure 3-9 is a demonstration of what occurs during this (gentle mixing results in the particle contact which enables this agglomeration to occur).

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[14] See appendix A-1 for the three basic equations that are used for calculating the ion distribution in the diffuse part of the electrical double layer. Refer to Hiemenz (1986) for models of the diffuse double layer.

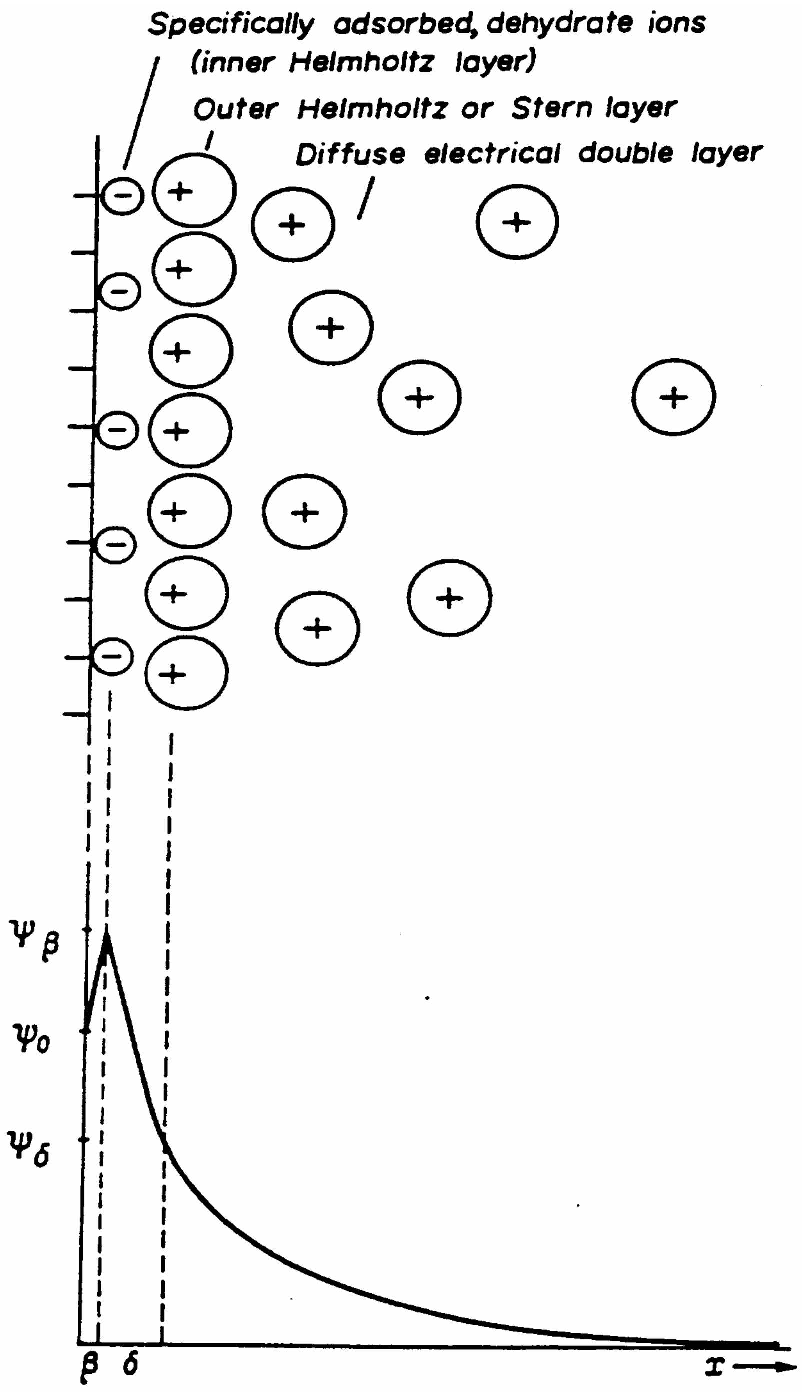

Figure 3-8: Counter-ion Distribution and Potential Drop v Distance[15]

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[15] Source: Sonntag et al., 1972

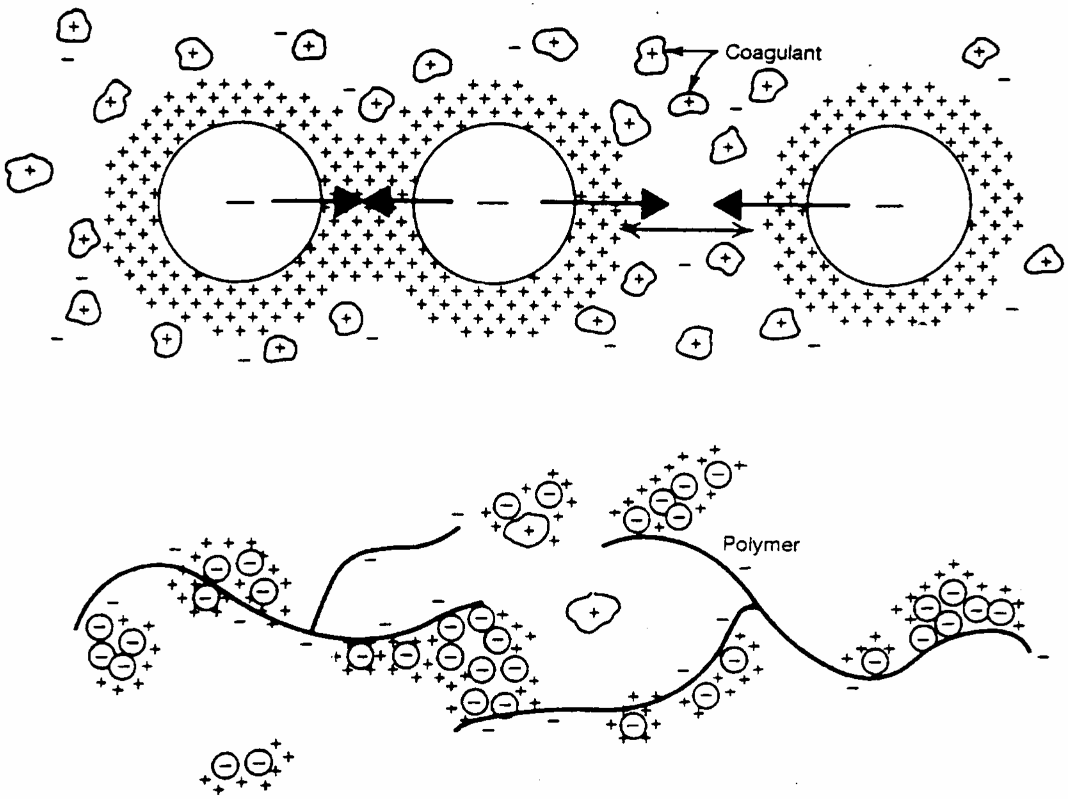

Figure 3-9: Double Layer Compression and Agglomeration

The upper picture in Figure 3-9 illustrates the compression of the double layer charge on colloids (destabilization) by the addition of chemical coagulants. The bottom picture in Figure 3-9 illustrates the agglomeration resulting from coagulation with metal salt and a polymer. (Hammer & Hammer, 1992)

Zeta (ζ) Potential

The ζ potential can be defined as the voltage difference between the bulk solution and the particle including the layers of ions that stick so tightly that they move with the particle. It exists at the plane of shear, lying somewhere in the diffuse double layer (Benefield *et al.*, 1982). Hammer & Hammer (1992) defines the zeta potential as the magnitude of the repulsive force developed by the charged double layer of ions attracted to a particle.

In essence, the zeta potential measures the charge of a colloidal particle. It is related to the stability of a colloidal suspension (see Figure 3-10). The greater the zeta potential, the greater the repulsion forces, therefore, the greater the stability (Reynolds *et al.*, 1996). Therefore, low zeta potentials are associated with systems of lower stability. The zeta potential can be defined by the following equation:

$$\zeta = \frac{4\pi q d}{D}. \qquad \text{(Reynolds \textit{et al.}, 1996)} \qquad \textbf{(3-2)}$$

Where,

ζ = Zeta potential

q = Charge per unit area

d = Thickness of the layer surrounding the shear surface through which the charge is effective

D = Dielectric constant of the liquid.

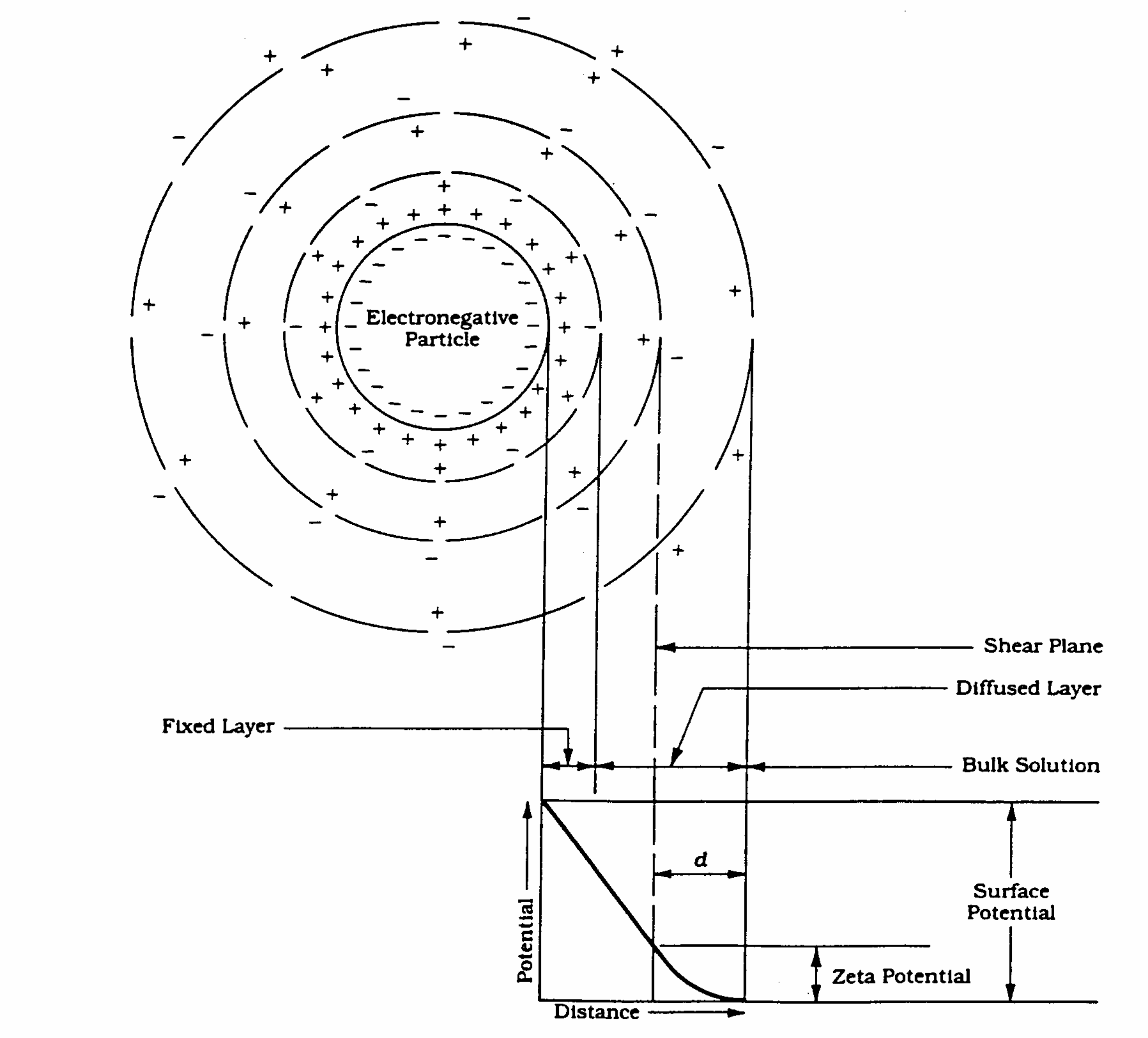

Figure 3-10: Negatively Charged Colloid and its Electrostatic Field[16]

Conclusion

It is these theories of coagulation and flocculation that are the key factors in jar tests and in the design of chemically enhanced facilities. That is, the design and efficiency predictions of Alternatives 2 and 3 (CEPT designs) are based upon this knowledge.

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[16] Source: Reynolds *et al.*, 1996.

Chapter 4 - Introduction to CEPT

Introduction

Chemically Enhanced Primary Treatment (CEPT) is the process by which chemicals (metal salts) and/or organic polyelectrolytes (polymers) are added to primary sedimentation basins to enhance the removal of solids (TSS) from wastewater via coagulation and flocculation. The chemicals typically involved include iron salts and/or aluminum salts (alum). The polymers utilized can be anionic, cationic, or non-ionic. The types of the metal salt coagulants and polymers, as well as the optimal dose are determined by performing jar tests at the intended wastewater site. CEPT will increase settling velocities and thus enhance the removal of suspended solids and their associated BOD. It will also result in a high removal of phosphorus and heavy metals.

The utilization of CEPT basically involves using a conventional primary treatment facility and adding chemicals to it [the addition of chemical coagulants to increase the efficiency of simple lagoons is called chemical precipitation in ponds, or, in-pond CEPT (Hanaeus, 1991)]. The metal salt (iron or aluminum) is typically added at the head of or upstream of the head of the plant to ensure good mixing and coagulation. The polymer, if added, should be added after the metal salt at a point where the mixing of the wastewater will not break-up the flocs, which are in the process of forming.

It is important to note that the chemicals added in CEPT are the same ones commonly added in potable water treatment, and that there is largely no residual iron or aluminum in the supernatant from the metal salts (Harleman & Murcott, 1992).

One of the key benefits of CEPT sedimentation basins is that they can be operated at overflow rates much greater than those of conventional primary settlers[17], while still maintaining a high removal rate of TSS and BOD. Operation at a high overflow rate allows for the construction of smaller basins, thus, a lower capital cost. CEPT also provides the opportunity for reductions in size of subsequent treatment units [or it can increase the capacity of existing conventional primary treatment plants], such as activated sludge basins when used in a combined primary and secondary biological treatment facility. Where odor is regulated, iron salts help control hydrogen sulfide.

In the case of a coastal city, CEPT is ideal since the removal of TSS is very high and the removal of BOD is sufficient so as to not impact oxygen concentrations in the ocean since the mixing is great. This is precisely the case in two of the largest operating CEPT facilities in the world (Point Loma, California, and Hong Kong). CEPT is also utilized for phosphorus removal by a number of facilities which discharge their effluent into the Great Lakes (Harleman & Murcott, 1992). A high removal rate of TSS is always desired due the adsorption of toxins to particulates. Thus, CEPT is also a "detoxifying" process. As noted, CEPT can remove a high amount of phosphorus, which can prevent the eutrophication of waters. Biological secondary treatment removes TSS and BOD at a very high efficiency, but does not effectively remove phosphorus, and produces nitrates (Morrissey, 1990). If this effluent does not undergo nutrient removal before it is released into a body of water, eutrophication can occur. The algal blooms often accompanying this kind of nutrient loading

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[17] The average overflow rate of conventional primary sedimentation basins ranges from 800 – 1200 gal/ft^2 (35 – 50 m/d) at average flow, whereas chemically enhanced primary sedimentation basins are often operated at overflow rate of 1500 – 2000 gal/ft^2 (60 – 80 m/d) at average flow.

will deprive the water body of oxygen, which would, in effect, be the same as releasing a high-BOD effluent into that body of water.

CEPT has been around for over one hundred years, yet it is not as commonly used as would be expected upon analysis of its performance. The notion was that CEPT utilized far too great an amount of coagulants and therefore incurred high costs and also dramatically increased sludge production. But, CEPT, most notably low-dose CEPT, is used today with minimal amounts of coagulants (10 - 50 mg/L). The theory of sludge increase is a misconception since the chemicals themselves make only a slight contribution to sludge production[18]. The greatest portion of the increase of sludge production is due to its increased efficiency of TSS removal in the primary clarifiers. This is the goal of CEPT, the increase of TSS removal, TP removal, and BOD removal, in the sedimentation process.

CEPT is also an effective means of preparing wastewater for disinfection. With its high removal of TSS, CEPT effluent can easily and effectively be disinfected via chlorination and ultra-violet irradiation.

There are many different methods to treat wastewater beyond primary and/or biological secondary treatment, whether or not CEPT is utilized. Some of these include gravity filtration, nutrient removal, high lime dosing, granular activated carbon, and reverse osmosis. These, when added following CEPT or to conventional primary and secondary biological treatment, can create a very clean effluent, much cleaner than CEPT or primary and

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[18] The amount produced by Alternative 2 is 460 kg/d on a dry weight basis (15% of the total sludge produced), and the amount produced by Alternative 3 is 275 kg/d on a dry weight basis (less than 10% of the total sludge produced). As will be discussed in detail in Chapter 6.

secondary biological treatment alone can achieve. But, due to their exorbitant costs (2 – 9 times as much as CEPT alone), they will not be considered as they are not viable options for developing countries (National Research Council, 1992). Just as important as deciding which form of liquid wastewater treatment to utilize for a facility is sludge handling and treatment, which is covered in Chapter 6.

CEPT is a relatively simple technology providing a very low cost, effective (high level of treatment), which is an easily implemented process (Harleman & Murcott, 1992).

Financial Benefits of CEPT

On top of what is already necessary for conventional primary treatment facilities and simple lagoon facilities (for in-pond CEPT), the addition of metal salts and/or a polymer will only require tanks for the chemicals and injection equipment. These expenses are very low, especially when compared to aerators for aerated lagoons. Aerators involve a large capital investment, and a great deal of maintenance, for its parts and for the cost of energy consumption. Therefore, eliminating the use of these aerators will reduce capital and maintenance costs (this is in reference to comparing Alternative 1 to Alternatives 2 and 3).

Table 4-1 presents data comparing the costs of primary treatment, secondary biological treatment, and chemically enhanced primary treatment.

Table 4-1: Comparison of Costs for Different Treatment Levels[19]

	CAPITAL COSTS ($/GPD)	O&M COSTS ($/MG)	TOTAL COSTS
Primary Treatment	0.9 – 1.1	205 – 240	450 – 550
Biological Secondary Treatment	2.4 – 2.6	320 – 410	930 – 1,130
Low Dose CEPT	1.1 – 1.4	230 – 280	550 – 680

This demonstrates how low-dose CEPT costs minimally more than primary treatment, and only about half as much as secondary treatment[20]. Yet, the removal efficiencies show CEPT's superiority, as discussed in the next section.

The implementation of CEPT in conjunction with lagoons will allow the use of smaller and/or fewer lagoons than is presently proposed for the CEAGESP site in Tatui, Brazil. Also, the use of aerators will no longer be necessary. And if used directly in the lagoons (in-pond CEPT), it will also increase the efficiency of TSS and BOD removal, and require no aerators and smaller subsequent lagoons.

Efficiency of CEPT

Table 4-2 displays the efficiencies of the aforementioned treatment schemes[21].

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[19] Source: National Research Council, 1992.

[20] Little data exists as to the cost comparison of in-pond CEPT treatment and is therefore not in Table 2-1.

[21] Again, the three processes represented in Table 4-2 are very common treatment methods which have been thoroughly studied and therefore have ample information on their efficiencies. However, data on the performance of in-pond CEPT lagoons and CEPT as pre-treatment is scant and cannot be put into this table. But, examples of their efficiencies are located in Chapter 7.

Table 4-2: Removal Efficiencies of Different Treatment Methods[22]

	TSS (%)	BOD (%)	TP (%)	TN (%)	FOG[23] (%)
Primary Treatment	55	35	20	15	51
Biological Secondary Treatment	91	85	30	31	98
Low-Dose CEPT	85	57	85	37	71

Table 4-2 illustrates how CEPT enhances the removal of TSS and its associated BOD; through chemical coagulation and flocculation, followed, of course, by settling of the floc.

The data show CEPT's superiority over conventional primary treatment. And, by incorporating the cost analysis in Table 4-1, its superiority over secondary biological treatment.

Ease of Implementation

A conventional primary treatment process is very simple, consisting of bar screens, a grit chamber, and primary clarifiers (see Figure 4-1). The implementation of an in-pond CEPT facility is even simpler since it involves a lagoon in lieu of the primary clarifiers. To upgrade a conventional primary treatment facility to a CEPT facility, basically all that is needed is the

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[22] Source: National Research Council, 1992.

[23] Fats, Oil, and Grease.

addition of a chemical coagulant (metal salt and/or polymer) at the inlet of the grit chamber and potentially a polymer at the inlet of the sedimentation basin (see Figure 4-2). In the present Tatui case and in the proposed reconstruction of the facility, only lagoons exist. Therefore, the bar screens, the grit chamber, and the sedimentation basin would need to be built. But, with its high surface overflow rate, the sedimentation basins will not need to be large (when compared to conventional primary sedimentation basins). And the use of rectangular sedimentation basins, as Alternative 2 proposes, will allow the use of common walls, which will reduce capital costs.

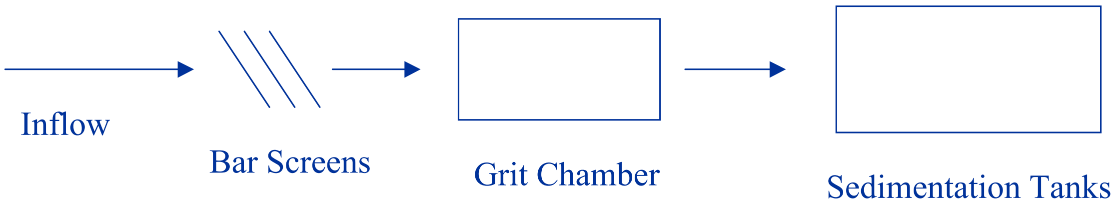

Figure 4-1: Schematic of Conventional Primary Treatment

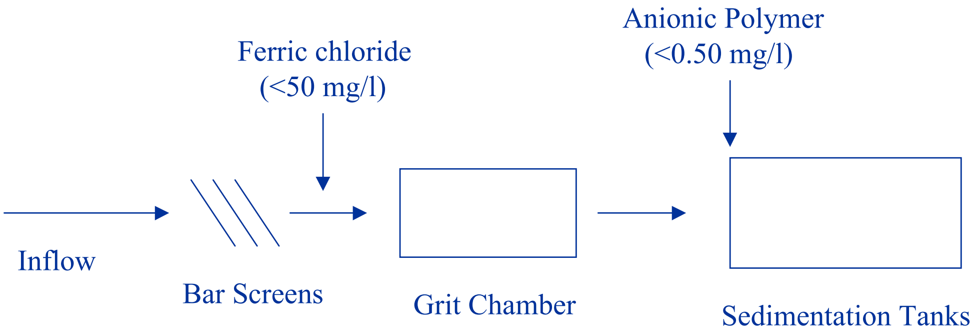

Figure 4-2: Schematic of CEPT Upgraded Primary Treatment[24]

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[24] For upgrading, refer to Harleman and Morrissey (19).

Coagulation and Flocculation

Coagulation and flocculation (discussed in Chapter 3) are the processes by which CEPT, pre-pond CEPT, and in-pond CEPT demonstrate their great efficiency over conventional primary treatment. It should be noted that the purpose of CEPT is not only to settle non-settleable particles (such as colloids). It is also the intention of CEPT to increase the settleability of all particles, which decreases their settling times, thus decreasing the size of the sedimentation basin(s), thus increasing the overflow rate, which in turn will increase treatment capacity.

Existing CEPT Facilities

See Table 4-3 and the following discussion for data and descriptions of CEPT facilities currently being operated in the USA.

Table 4-3: Characteristics of Existing CEPT Facilities

PLANT	% REMOVAL (YEARLY AVERAGES)					COAGULANT DOSAGE [mg/L]	POLYMER DOSAGE [mg/L]
	TSS	BOD_5	FOG	TKN	PHOSPHORUS		
Point Loma	86	59	70	-	92	25	0.1
Orange County	75	50	53	15.4	NA	20	0.2
JWPCP	78	42	-	-	-	0	0.15
Hyperion	88	54	-	-	-	6	0.08

Point Loma

The Point Loma Plant is located in (Point Loma) San Diego, CA and is one of the largest operating CEPT plants in the world. Point Loma serves 1.8 million citizens in Southern California and treats approximately 187 million gallons per day (MGD), with a peak handling capacity of 240 MGD (910,000 m^3/d). The facility is currently undergoing reconstruction/expansion, which will add a couple of sedimentation basins, and anaerobic sludge digesters.

The Point Loma facility was given a waiver from secondary biological treatment since it is a coastal city and its utilization of CEPT gave it a high quality effluent suitable for discharge to the Pacific Ocean. The California Ocean Plan, which Point Loma abides by, calls for a process in which the effluent would not affect the ocean's dissolved oxygen concentration by more than 10% below ambient levels (Harleman & Murcott, 1992). Table 2- presents the characteristics of Point Loma's CEPT facility.

Table 4-4: Point Loma Influent and Effluent Characteristics in 1998

Parameter	Influent Concentration (mg/L)	Effluent Concentration (mg/L)	%Removal
TSS	284	39	86
BOD_5	259	106	59
TP	6.2	0.5	92
FOG	32.3	9.9	70

As Table 4-4 indicates, Point Loma has a high removal efficiency of TSS, TP, and FOG. The removal efficiency of BOD_5 is not as high, compared to full conventional primary plus secondary biological treatment facilities, but is sufficient to be in compliance of the California Ocean Plan.

Point Loma's facility includes bar screens, followed by an aerated grit chamber, followed by rectangular chemically enhanced primary sedimentation basins (12 basins). The influent receives sodium hydroxide (NaOH) and hydrogen peroxide (H_2O_2), to control odor problems. In the grit chamber, the collected grit is dewatered by a cyclone separator. The separated grit is hauled off to a hazardous waste landfill in Arizona, and the supernatant is returned to the head of the plant. The iron salt ($FeCl_3$ at a dosage of 25 mg/L) is added before the aerated grit chamber, which maintains a velocity of 2 ft/s. The $FeCl_3$ is stored in two 10,000-gallon tanks, which are refilled about two times daily, and fed to the influent by 2-horsepower centrifugal pumps. The polymer (anionic) is added at the head of the rectangular sedimentation basins at a dosage of 0.10 mg/L. The polymer is stored in a tank of a capacity of 6500 gallons which is refilled once every three to four days. The polymer is pumped to a smaller tank where it is made up and then pumped to the flumes of the primary clarifiers for injection. The 12 rectangular sedimentation basins have an average detention time of 1.5 hours at average flow, and contain three cells per basin. The basins are equipped with baffles to ensure horizontal flow, although short-circuiting still occurs. A revolving rake collects the sludge and floating scum. Their average overflow rates are about 2000 gpd/ft^2 (81.4 m/d), with a range of 670 – 2411 gpd/ft^2, depending on the flow.

Six anaerobic digesters, following a two-stage process, are currently treating the sludge. Four of them are first-stage digesters, and the subsequent two are the second-stage digesters.

The sludge residence time for the first stage is approximately 15 days, and 3-5 days in the second stage. Each digester has a volume of approximately 4 million gallons, which process a total of 145 dry tons of sludge per day. The methane (CH_4) produced from the anaerobic digesters is currently being burned off due to the reconstruction of the plant which recently tore down the generation facility in order to a build newer and more efficient generator facility. Upon completion of reconstruction, the CH_4 will be utilized to generate power for the whole plant, with the excess being sold to the local power utility (San Diego Gas & Electric). This power generated is also used to heat the sludge prior to entering the anaerobic digesters in order to make the influent sludge the same temperature as the sludge present in the digesters.

Operational and maintenance problems, as with all wastewater treatment facilities, do exist. The sedimentation basins are cleaned and scheduled for other maintenance periodically, usually every three months, with a major overhaul once a year. The basins are only shut down if a major problem comes up. Many little problems exist, such as failure of chemical pumps and chains for the sedimentation basins. The main maintenance involved with the anaerobic digesters is associated with the influent sludge heaters.

The plant's effluent is screened to remove any grease that may pass through the system, and is not chlorinated. The ocean outfall at Point Loma, as in many other plants, is discharged at a great distance from the coast (here, 4.5 miles), and at that point is dispersed through diffusers.

OCSD

The Orange County Sanitation District (OCSD)[25] in Orange County, California utilizes CEPT on 100% of its influent, in which 50% of the advanced primary effluent undergoes secondary biological treatment (which takes place in two plants). The remaining 50% are discharged directly into the ocean through its ocean outfall with effluent from secondary treatment. The data provided in Table 4-5 is the OCSD's CEPT data only, and therefore does not include the effects of the secondary biological treatment. The primary basin design overflow rates are 700 gpd/ft^2 (30 m/d). The amount of wastewater treated is an average of 240 MGD. The outfall is five miles long and has a one-mile long diffuser on the end of it with 500 small (2-inch) ports.

Table 4-5: OCSD Influent and Effluent Characteristics in 1998

Parameter	Influent Concentration (mg/L)	Effluent Concentration (mg/L)	%Removal
TSS	240	60	75
BOD_5	230	115	50
TKN	39	33	15.4
FOG	51.2	24.3	53

The coagulant used is ferric chloride ($FeCl_3$), dosed at 20 mg/L which is fed upstream of the grit chamber. The polymer used is an anionic polymer, dosed at 0.2 mg/L, which is fed at the

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[25] The data on OCSD was provided through contact with Mr. Robert Ooten from the facility.

inlet to the primary clarifiers. Note, again, that all of the data presented is for the efficiency of the chemically enhanced primary clarifiers only, that is, not including the performance of OCSD's secondary biological treatment.

JWPCP[26]

The Joint Water Pollution Control Plant (JWPCP) in Los Angeles County, CA treats an average of 380 MGD (1.13 Mm^3/d). The plant uses CEPT in conjunction with secondary biological treatment. Here, approximately 60% of the CEPT effluent undergo secondary biological treatment; the combined effluent is discharged into the Pacific Ocean. JWPCP has two outfalls. One outfall is approximately 2250-m long, 60-m deep and equipped with a 1340-m diffuser. The second outfall is approximately 2500-m long, 60-m deep and equipped with a y-shaped diffuser with each arm extending approximately 670m. The plant operates at an overflow rate of greater than 1300 gpd/ft^2 (53 m/d) (Morrissey, 1990).

HTP[27]

Hyperion Treatment Plant (HTP) is located in Playa Del Rey, CA. HTP serves over 4 million customers from the Los Angeles area and treats an average of 360 MGD (1.36 Mm^3/d). The plant is a CEPT and (soon to be) full secondary facility.

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[26] Data on JWPCP obtained from Morrissey, 1990.

[27] Data provided via conversation with Mr. Mike Noguchi and Y.J. Shao of HTP (March, 1999).

The average influent BOD is 290 mg/L, with an average effluent BOD of 135 mg/L. The influent TSS is 320 mg/L, with an effluent TSS of 40 mg/L. The chemical coagulant at HTP is dosed upstream of the plant.

Conclusion

CEPT is an efficient, cost-effective and easily implemented wastewater treatment technology, based on data from large-scale municipal wastewater treatment plants in Southern California with multiple years of operating experience under a CEPT regime. The addition of chemical coagulants and/or polyelectrolytes allows for the increased removal of total phosphorus, total suspended solids and its associated biochemical oxygen demand. The increased removal of TP, TSS and BOD is accompanied by increased settling rates of particles, which allows for the design of smaller basins and greater overflow rates.

Chapter 5 - Jar Tests & Chemical Analysis

Introduction

The purpose of this chapter is to demonstrate the analysis and results of the jar tests and its accompanying data (which is located in Appendix C).

CEPT is the addition of chemical coagulants and polymers to enhance the removal of TSS and its associated BOD_5 and TP. Therefore, it is important to evaluate and analyze the chemicals utilized, as well as the analysis of the data obtained.

Methods and Procedures

The six parameters analyzed in the CEAGESP facility were pH, TSS, chemical oxygen demand (COD), total phosphates as PO_4—(TP), total sulfates as SO_4^{2-}, and temperature. The parameters were chosen based upon the following factors: due to time constraints and technological deficiencies of the lab, BOD_5 was not able to be determined; the parameters are representative of the behavior of the lagoon and the efficiency of the chemicals/polymers utilized in the jar tests; and because many of these parameters (pH and temperature) indicate the condition of the present system. It should be noted that jar tests slightly over-predict TSS removal (Harleman & Murcott, 1992)

Laboratory Study and Setup

The purpose of the field trip was to asses the efficiency of the CEAGESP treatment lagoon system in Tatui, and to conduct jar tests on the raw influent to predict the efficiency of a proposed CEPT treatment plant option, as well as selecting the optimum chemicals and dosages.

The tests conducted in Tatui concentrated on assessing the COD, and TSS of samples from both the pond system and the jar tests.

The laboratory was set-up in an old motorcycle repair shop on the outskirts of Tatui. A trip to the lagoon system for sampling took approximately ten minutes. Although only temporary, the lab was extremely well arranged, and work could be carried out efficiently. Figure 5-1 presents the lab, as seen from the outside.

Figure 5-1: The Laboratory in Tatui

The lab was divided into three sub-sections: the raw sewage section, the jar-test bench, and the TSS and COD analysis bench. Forty liters of raw sewage were collected every morning at the splitter box and were transported back to the lab, where they were dumped in a large plastic garbage can. This amount of raw sewage was necessary to do a day's worth of jar testing.

Jar Tests

The jar test is a common laboratory procedure, which will be used to determine, empirically, the optimum operating condition for Alternatives 2 and 3. The jar test apparatus used was manufactured by Phipps & Bird, as shown in Figure 5-2.

Figure 5-2: Phipps & Bird Jar Test Apparatus

The jar test procedure is presented in Table 5-1. The table shoes the chief mixing regime used. Other regimes were sometimes used, such as when no polymer was added, the second 100-rpm mixing for 30 seconds stage was omitted. Also, different mixing regimes were used, such as when the recycling of chemical sludge was tested.

Table 5-1: Jar Test Mixing Regime

Steps	Mixing Intensity (rpm)	Mixing time
Raw water	100	15 sec
Primary Coagulant Added (metal salt)	100	30 sec
Polymer added	100	30 sec
Medium mixing	70	2.5 min
Slow mixing	30	2.5 min
Settling	0	5 min

During the slow mixing and settling phases of each jar test run, each jar was analyzed to visually characterize floc size and any other observation in each jar. At about 4.75 minutes into the 5-minute settling approximately 20ml of supernatant was removed, in order to flush the sampling tube. At the five-minute mark, a sample of approximately 150-200ml was taken from each jar and subsequently used in COD and/or TSS analyses. However, it should be noted that COD and TSS analyses were not done for every sample.

COD Analysis

The COD test measures the oxygen equivalent of the organic matter in a wastewater sample that can be oxidized chemically using dichromate in an acid solution ((Franzini *et al*., 1992). COD was measured using the Hach's adaptation of Standard Methods. Two milliliters of sample were pipetted into the Hach COD vials. These Hach test tubes contained a COD reagent as per Hach's adaptation of Standard Method. The mixture was vigorously shaken before being cooked in the Hach COD reactor for two hours at 150°C. The cooked samples were then placed in the Hach spectrophotometer (Figure 5-3) for the COD reading.

Figure 5-3: Hach Spectrophotometer

The Hach COD method is, as can be seen above, extremely simple. Also, this method is very convenient in assessing (chemical) oxygen demand. Although COD was used, and is a good indicator, it can not be directly converted into a BOD value. As stated earlier, COD is converted to a BOD value, but over a range. This is so because of several reasons. One is that many organics can be oxidized chemically, but cannot be oxidized biologically. Another reason is that some inorganic substances are oxidized by the dichromate, and increase the apparent organic content of the sample. A third reason is that wastewater often contains a host of toxic substances, which often are toxic to the microorganisms utilized in a BOD analysis. One last reason is that substances often interfere with the reading of COD (Franzini *et al*., 1992).

Although the BOD_5 test is commonly used for the purpose of determining oxygen demand, it suffers from several serious deficiencies. The most serious deficiency in the BOD_5 test is that it has no stoicheometric validity. That is, the arbitrary 5-day period usually does not correspond to the point where all of the organics are consumed. The 5-day value is actually used because the test was developed in England where the maximum time of flow of most rivers from headwaters to the ocean is 5 days (Franzini *et al.*, 1992). Another factor in not choosing to do the BOD_5 test is that nitrification (nitrogenous oxygen demand) often interferes with BOD_5 data if the nitrogen is not treated properly. Moreover, a BOD_5 test necessitates a constant 20^oC temperature of sample for the five days, a condition that is hard to attain in makeshift labs such as the one used in Tatui. Typical ratios of BOD_5 to COD for untreated wastewater are 0.50-0.65 and for primary settled water is 0.40-0.55. (Franzini *et al.*, 1992).

TSS Analysis

TSS were measured using Standard Methods. A 47-mm diameter glass fiber filter, which had been previously weighed, was introduced on the filtration apparatus (see Figure 5-4). The plastic cup was then placed onto the filtration apparatus. The vacuum pump was then turned on, and the filter was wet with 10 ml of distilled water. A sample of 25 ml or 50 ml (depending on the clarity of the sample) was then introduced in the plastic receptacle. Thirty seconds later, 10 ml of distilled water were introduced in the plastic receptacle to wash out the receptacle. The same amount of water was introduced 30 seconds later and 1 minute later.

Figure 5-4: TSS Filtering Setup

The filter was then removed from the plastic support and put in its aluminum cup. The filters were then dried for two hours at 103°C in the oven, cooled, and then weighed.

Lagoon Sampling

The stay in Tatui involved seven days of lagoon sampling. The following section will describe the CEAGESP lagoon system in Tatui, and will explain the sampling procedure.

Figure 5-5: Panorama of the CEAGESP Lagoons (Anaerobic Pond)

The CEAGESP lagoons are located approximately 7 kilometers southwest of Tatui. The lagoons are in an agricultural area, and there are two redbrick factories two kilometers up-road. The Tatui River and a densely forested hill border the lagoons on the eastern side. A railroad track runs on the western side of the lagoons, between the lagoons and the dirt road (see Figure 5-6).

A typical sampling exercise would start by measuring the height of water flow through the Parshall flume. The group would then proceed by taking a forty-liter sample out of the splitter box. The sampling of these 40L would be carried out by using a bucket to collect raw sewage, and dumping its contents into one of four 10L jugs. Smaller 1L samples would also be taken at the outlet weir of the anaerobic pond and at the outlet weir of the facultative pond. Temperature and pH were measured at each sampling point.

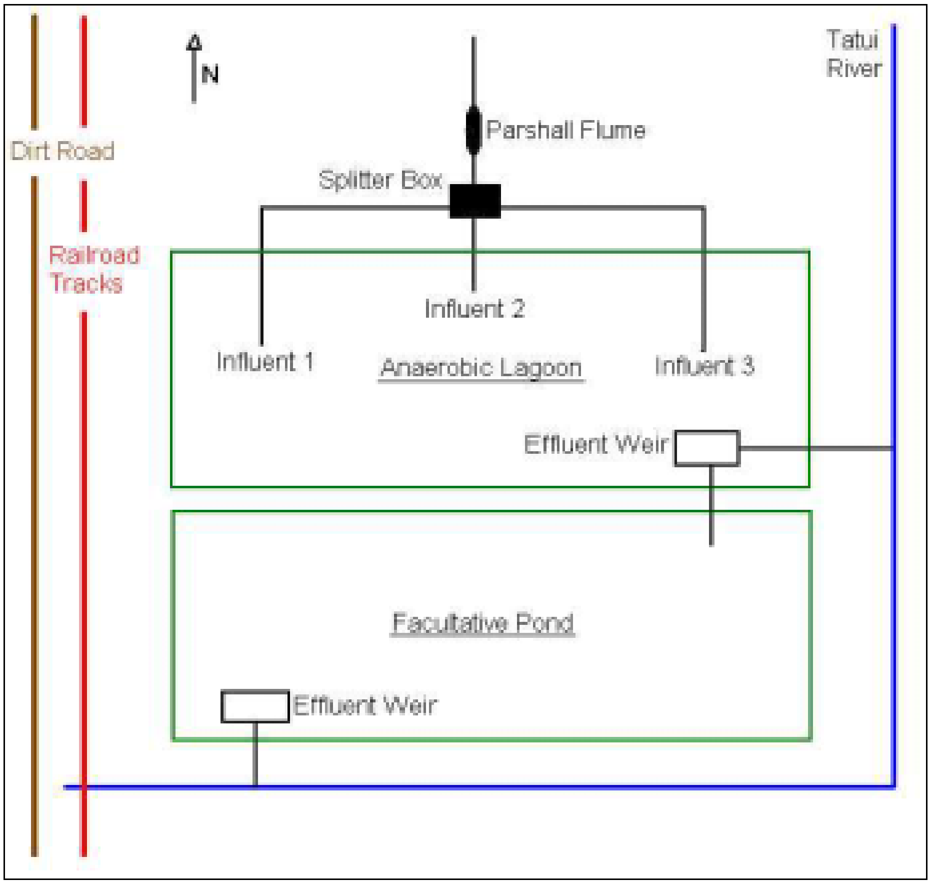

Figure 5-6: Schematic of the CEAGESP Lagoon System

24-hr Lagoon Test

Part of the lagoon sampling entailed a 24-hr sampling test to assess the diurnal variations of the influent characteristics and the performance of the lagoon systems. Table 5-2 displays the results from the 24-hr sampling test.

Table 5-2: 24-Hour Lagoon Sampling Data

	Anaerobic Lagoon	Facultative Lagoon	Total System
COD %Removal	35.5	26.2	52
TSS %Removal	25.3	43.3	54.5

Chemical Coagulants

The chemical coagulants utilized in the jar tests were ferric sulfates and ferric chlorides. These were the main coagulants tested, although, a few aluminum salts were also tested in compliance with SABESP's requests. The chemical coagulants will be referred to by their commercial names: Sanechlor, Kemwater, NHEEL, Liex, and Eaglebrook.

Sanechlor

Sanechlor jar tests were run on the 16th and 17th of January. This chemical is produced in Sao Paulo, Brazil by Sanechlor Produtos Quimicos Ltda. This chemical coagulant is 40% ferric sulfate (solids) by weight and is 11% iron by weight.

As the results show in Appendix C, a satisfactory removal of COD and TSS are achieved by a dosage of Sanechlor higher than that of the three ferric chlorides tested. Although the dosage is higher, the price of the chemical is approximately 60% of the ferric chlorides (a major factor in selecting the best chemical coagulant). The concentration range of 40 mg/L to 60 mg/L represents a removal efficiency of 38%-67% of COD and 64%-94% removal of TSS. To determine the ideal combination and dosage of chemical coagulant with polymer, a jar test was run with 46 mg/L of Sanechlor and a polymer dosage of 0.25 mg/L. This jar test removed 92% of TSS, but due to lack of COD reagent, a COD analysis was not performed. A problem with Sanechlor, as with Kemwater and all other sulfate-containing chemical coagulants, is the sulfate (SO_4^{2-}) in the compound. The sulfate in Sanechlor disassociates from the $FeClSO_4$ and upon entering anaerobic conditions is oxidized and forms hydrogen sulfide (H_2S), thus increasing odor problems, which would most definitely occur in Alternative 3.

Due to the fact that a larger dosage of Sanechlor is necessary to achieve removal efficiencies equivalent to that of the ferric chlorides tested, the use of Sanechlor would necessitate a larger storage tank, and a pump with a larger capacity. This, combined with its containing sulfate and its jar test performance, was influential in Sanechlor's elimination from being the optimum coagulant. Full data on Sanechlor is in Appendix C.

NHEEL

NHEEL jar tests were run on the 16th and 17th of January. This chemical is produced in Brazil. This chemical coagulant is 38% ferric chloride (solids) by weight and is 34% iron by weight.

As is evident upon inspection of Appendix C, NHEEL performed very well. Its ideal dosage was between 40 mg/L and 50 mg/L, which achieved COD removal efficiencies of 57%-68% and TSS removal efficiencies of 84%-96%.

NHEEL was chosen as the optimum coagulant, at a dosage of 50 mg/L for Alternative 2, and at a dosage of 30 mg/L for Alternative 3. Alternative 2 necessitates a larger dosage because the influent has a detention time of 1.2 hours, whereas Alternative 3 has a residence time on the order of one to two days. This dosage range produced a clear supernatant and large flocs. [For full data on NHEEL, see Appendix C.]

Eaglebrook

The Eaglebrook jar tests were run on January 15th and 17th. This chemical is produced in Schereville, IN by Eaglebrook, Inc. This ferric chloride is 40% solids and 34% iron.

From analysis observations, in conjunction with visual observations, Eaglebrook formed the best floc (largest). It achieved COD and TSS removal efficiencies of 60%-72% and 92%-96%, respectively, at dosages of 40 mg/L to 50 mg/L. In the cost analysis, however, Eaglebrook was not a viable choice as the optimum chemical coagulant since it is not currently produced (or even available) in Brazil, and there is no information as to how much it would cost if provided in Brazil. The chemical was tested in Tatui, Brazil because it is a well-known chemical coagulant in the United States with a reputation of achieving high removal efficiencies. It served as a measuring stick for the performance of the other ferric chlorides. [For full data on Eaglebrook, see Appendix C.]

Liex

As with NHEEL, so too is Liex a ferric chloride produced in Brazil. It contains 40% solids, and is 34% iron. The Liex jar tests were run on January 15th and 17th.

Liex achieved COD removal efficiencies of 62%-71% and TSS removal efficiencies of 76%-82%, with dosages of 40 mg/L to 50 mg/L. Liex performed very similar to NHEEL, and is the same price. But, Liex formed smaller flocs and had inferior COD and TSS removal efficiencies when compared to NHEEL. Furthermore, NHEEL outperformed Liex in the side-by-side coagulant comparison of January 16th. For these reasons ijt was not chosen as the primary coagulant. [For full data on Liex, see Appendix C.]

Kemwater

Kemwater is a ferric sulfate produced in Brazil by Kemwater Brazil S.A. It contains 43% solids, and has a density of 1.58 g/cm^3. Kemwater was tested in jar test on January 15th and 17th.

Kemwater achieved the lowest removal efficiencies of all of the chemical coagulants. It achieved removal efficiencies of 40% for COD and 48% for TSS at a dosage of 60 mg/L. It also was inferior to all other chemicals in visual observations of flocs formed and supernatant clarity. A direct comparison of Kemwater to Sanechlor is possible since they are a similar type of coagulant and cost the same amount in Brazil. Since Kemwater was evidently inferior to Sanechlor, it too was not chosen as the optimum coagulant. As stated in the sectio0n describing Sanechlor, the Kemwater product we tested contains sulfates. It is $Fe_2(SO_4)_3$, thus it will release three sulfates for each $Fe_2(SO_4)_3$ added to the wastewater. Again, this will increase odor problems through H_2S production. [For full data on Kemwater, see Appendix C.]

Chemical Sludge Recycling as a Coagulant

Facilities do exist in which chemical sludge from CEPT is used as a coagulant, most notably in France. The jar tests conducted in Tatui tested the use of chemical sludge, collected on different occasions with different circumstances and different mixing regimes. The first time chemical sludge was collected was on January 16th where NHEEL was the chemical used. The mixing regime utilized for this was as follows: mix the raw sample for 15 seconds at 100 rpm, then reduce the mixing speed to 30 rpm and add the chemical sludge and mix for 2.5 minutes, then allow the sample to settle (that is, no mixing) for 5 minutes and take samples for further analysis. This jar test displayed poor COD removal efficiencies (a maximum of 27% at a chemical sludge dosage of 60 mg/L) and fair TSS removal efficiencies (a maximum of 72% at a dosage of 60 mg/L of chemical sludge). The mixing speed of 30 rpm was violent enough to break up the floc in the chemical sludge, thus yielding a poor performance. The next jar test run with chemical sludge was also on January 16th, immediately after the one explained above. This jar test involved the addition of NHEEL and NHEEL-generated sludge. The mixing regime was as follows: mix the raw samples at 100 rpm for 15 seconds; next add the predetermined dosage of NHEEL to the samples and stir at 100 rpm for 30 seconds; then stir the samples for 2.5 minutes at 70 rpm; next, stir the samples at 30 rpm for 2.5 minutes; now, reduce the mixing speed to 15 rpm, add the chemical sludge, and maintain the same mixing speed for another 2.5 minutes; last, allow the sample to settle for five minutes and take sample for TSS and COD measurements. This jar test yielded better results than the previous one with chemical sludge, as would be expected due to the addition of NHEEL and a lower mixing speed for the chemical sludge to prevent flocs from breaking. Though the results were better than those of chemical sludge addition alone, they did not perform well compared to equivalent dosages of NHEEL without the addition of chemical sludge. The last jar test run on chemical sludge was performed on January 21st, in which Sanechlor-generated sludge was produced and used with Sanechlor and polymer #17. The mixing regime was as follows: mix the raw samples at 100 rpm for 15 seconds; next add the predetermined dosage of Sanechlor to the samples and stir at 100 rpm for 30 seconds; next, add the 0.10 mg/L of polymer # 17 and stir the sample again at 100 rpm for 30 seconds; then stir the samples for 2.5 minutes at 70 rpm; next, reduce the mixing speed to 30 rpm, add the chemical sludge, and maintain this mixing speed for 2.5 minutes; last, allow the sample to settle for five minutes and take sample for TSS and COD measurements. Due to lack of supplies, only COD tests were conducted, with no raw sample to compare removal efficiencies to. Yet, the direct readings of COD are sufficient for a comparison. This jar test run involved two jars with the above procedure, the third jar did not receive chemical sludge. As is evident in Appendix C, jar #3 (ID# 187), which received no chemical sludge, performed the best. Thus, it was decided that the recycling of chemical sludge will not enhance the treatment of Tatui's wastewater.

Other Chemical Coagulants

Various other chemical coagulants were used in the jar tests. Coagulants such as Alum were used to make observations as to their applicability to Alternatives 2 and 3 for CEAGESP. The performance of these chemicals is in Appendix C. Since none performed well, no further analysis was undertaken.

Lime was not chosen as a candidate for the optimum chemical coagulant. One of the reasons is because lime coagulants generate huge amounts of sludge. Another reason is that lime forms particles with settling velocities that are too fast, and thus can clog the inlet pipes to the CEPT lagoon or the CEPT basins (Hanaeus, 1991). Also, Alternatives 2 and 3 will not dose the coagulant for 24 hours which can pose a problem for lime dosing facilities. When lime dosing terminates, the pH will drop substantially and this change in pH will bring parts of the lime sludge into solution. Indeed, lime could be analyzed for other non-lagoon facilities, but for lagoon facilities with non-24-hour dosing, it is not an optimum choice.

Polyelectrolytes

Polymers are added to wastewater to improve the settleability of solids through particle bridging. Four main polymers were used for the Tatui Jar Tests. Other polymers (non-ionic and cationic) were provided by SABESP, but did not perform well and thus data was not collected on them. The main polymers tested, on January 19th, were polymers #13, #15, #17, and #19 produced by the General Alum & Chemical Corporation of Maine. The chemical coagulant used in each jar test is present in Appendix C.

It should be noted that no tests were undertaken to determine the effect of only polymers on the removal of TSS and its associated BOD. Thus no comparison (qualitative or quantitative) can be made between dosing only polymer versus polymer with coagulant or dosing just coagulant. But, it was visually observed that polymers dramatically inrease floc size and dramatically decrease settling times.

Polymer #13

Polymer #13 is an anionic polymer with a low (10%) charge density and a high molecular weight (6 million Daltons). This polymer performed the worst, as can be seen in Appendix C, and was thus not used as the ideal polymer.

Polymer #15

Polymer #15 is an anionic polymer with a high (40%) charge density and a high molecular weight (6 million Daltons). Polymer #15 performed very well, approximately as well as polymer #17, but in the comparison tests of polymer #15 versus polymer #17, polymer

#17performed better (according to visual observation). For the data on polymer #15, see Appendix C.

Polymer #17

Polymer #17 is an anionic polymer with a medium charge density (20%) and a medium molecular weight (4 million Daltons). As stated above, polymer #17 outperformed polymer #15 in “head-to-head” jar tests, and was subsequently chosen as the best polymer based on limited testing. For observations from January 19th for this polymer, see Appendix C. Since polymer #17 was chosen as the best polymer, it was used on January 20th in a comparison test between Sanechlor and NHEEL to determine the optimum coagulant-polymer combination. Then, on January 21st, it was used to determine the ideal polymer and coagulation dosage. As stated above (and below), this was not determined due to time and equipment restraints.

It can not be said for the obtained results that a metal salt/polymer combination is recommended; only that certain polymers out-performed others.

Polymer #19

Polymer #19 is an anionic polymer with a high charge density (40%) and a very high molecular weight (8 million Daltons). As can be seen in Appendix C, polymer #19 performed rather poorly and was subsequently not chosen as the optimum polymer.

Other Polymers

As stated, SABESP provided the MIT group with different polymers (cationic and non-ionic) which they were interested in testing. The performance of these polymers was poor and was thus not analyzed further in jar tests. These polymers were probably used by SABESP for sludge thickening and were not the optimal ones for enhancing flocculation. The performance of these polymers can be found in Appendix C.

Optimum Polymer Dosage

One of the objectives of a jar test is to determine the optimum coagulant-polymer combination and the optimum dosages of both.

Due to time and supplies constraints, it was not possible to determine the optimum dosage and combination of chemical coagulant with polymer. Moreover, a metal salt coagulant/polymer system is somewhat more complex and expensive than a singe coagulant system. Therefore, there will be no polymer dosing for the design of the CEAGESP facility in Tatui. The only enhancement to the wastewater process will be metal salt addition (specifically, NHEEL).

Chapter 6 - Sludge

Introduction

The purpose of this chapter is to introduce and explain several options for the processing, treatment and disposal of sludge (also known as bio-solids), followed by an analysis of the chosen method(s) for the Tatui design.

For the purposes of this paper, sludge is that which settles in sedimentation basins and lagoons. This is not to be confused with that which settles in a grit chamber (conveniently called grit).

The processing, treatment and handling of sludge is one of the most difficult aspects of wastewater treatment and it still perplexes engineers today. The complexity stems from the fact that sludge is the reason wastewater is considered so offensive, and because sludge is mostly water (0.5% – 12% dry solids).

The amount of sludge created by CEPT is dependent upon the removal of TSS, TP, BOD_5, and the amount of chemical coagulant added, although, the chemical addition and TP removal will add only a small amount to the total sludge. Therefore, it is commonly found that as the removal efficiencies of TSS and BOD_5 increase, so will sludge production.

Many options exist as to what to do with sludge. Sludge can be used for making bricks, it can be composted, land-filled, directly applied to land (if treated properly) and it can be incinerated. This chapter will only deal with more established sludge handling processes, excluding innovative approaches. Calculating the amount of sludge produced from TSS,

BOD_5, TP and chemical addition is quite simple; yet, what to do with the sludge after it is collected is the challenge.

Sludge processes include thickening, stabilization, anaerobic sludge digestion, aerobic sludge digestion, composting, conditioning, disinfection, dewatering, heat drying, thermal reduction, and land application. Dewatering, conditioning, and thickening are mechanisms which have the primary purpose of removing water from the sludge, to increase the %dry solids content. Digestion (whose purpose is to convert the organic constituents in raw sludge into a methane gas and a relatively inert material that can be quickly dewatered without the associated odors of raw sludge (Hammer & Hammer, 1992)), composting, and incineration are examples of processes to stabilize and/or treat the sludge.

Sludge comes from a variety of unit processes in a wastewater treatment facility. Screening produces coarse solids. Grit removal and preaeration produce grit (heavier inorganic solids such as coffee grounds, sand, and eggshells) and scum (floatable materials, including grease, food wastes, hair, paper, cotton, cigarette tips, condoms etc.). Primary sedimentation basins produce scum and primary sludge. Biological secondary treatment produces suspended bio-solids from the process of reducing BOD, while secondary sedimentation produces scum and secondary sludge. Sludge-processing facilities produce sludge, ashes, and compost (Metcalf & Eddy, 1991).

Preliminary Operations

The improved treatability of sludge is achieved through preliminary operations. The processing and disposal of sludge may involve many operations. These include sludge grinding, sludge degritting, sludge blending, and sludge storage. Sludge grinding is utilized to turn large particles into smaller ones in order to prevent damage to rotating equipment, which often require high maintenance. Sludge degritting of primary sludge is often used at wastewater facilities which do not have grit chambers, or have grit chambers that can not effectively remove grit at peak conditions. The sludge is degritted to allow for further processing of the primary sludge (again, the grit posing a threat to damage parts due to their abrasive characteristics). Sludge blending is utilized to produce a uniform sludge for further processing (Metcalf & Eddy). Sludge storage is utilized to handle peak flows of sludge and to store sludge when sludge-processing facilities are "down." Sludge storage is often done in settling tanks and digesters.

Thickening (Concentration)

The thickening of sludge removes water/moisture from sludge in order to increase the solids content of the sludge, thus decreasing the volume of sludge. Sludge is thickened to enhance subsequent sludge treatment processes, such as digestion, drying and combustion. A thickened sludge has a smaller volume which requires smaller tanks in subsequent treatment, reducing capital costs of those unit processes. Thickening is achieved via gravity thickening, flotation thickening, centrifugal thickening, gravity belt thickening, and rotary drum thickening.

Gravity thickening involves pumping the sludge into a tank similar to (circular) primary sedimentation tanks. The sludge settles by gravity, leaving a layer of supernatant on top of the sludge. The settled sludge is piped from the bottom; the supernatant is decanted and returned to the head of the plant.

Flotation thickening has the following three variations: dissolved-air flotation, vacuum flotation, and dispersed-air flotation. Dissolved-air flotation involves diffusing fine bubbles of air at the bottom of the tank into the dilute sludge. The air attaches to sludge, thus causing it to float, where it is subsequently removed (Hammer & Hammer 1992).

Centrifugal thickening uses a centrifuge to thicken and dewater the sludge. This application is limited in scope in that it can be used effectively only on waste activated sludge.

Gravity belt thickening facilities are similar to belt filter presses. The sludge is evenly distributed along a moving belt, on which the water drains through. The belt is cleaned after each cycle of use (Metcalf & Eddy, 1991).

Rotary drum thickening uses rotating cylindrical screens to thicken the sludge, which must be treated with a polymer prior to thickening (Metcalf & Eddy, 1991).

Stabilization

The stabilization of sludge reduces pathogens, eliminates the offensive odors associated with sludge, reduces the organic content of the sludge, and inhibits or reduces the putrefaction potential (Franzini *et al.*, 1992). Stabilization is commonly achieved through lime stabilization and heat treatment. In designing stabilization processes, it is important to keep in mind the amount of sludge necessitating stabilization, its effect on other processes used with it, and the objective of the stabilization process.

Lime Stabilization

Lime stabilization is achieved by the addition of lime to untreated sludge until the pH is greater than or equal to 12. This pH is sufficient enough to kill pathogens and microorganisms, and prevents odors and putrefaction. There are many sources of lime; either quick lime (CaO), hydrated lime ($Ca(OH)_2$), or a lime substitute can be used. Substitutes include fly ash, cement kiln dust, and carbide lime (Metcalf & Eddy, 1991). Lime dosing depends on the characteristics of the sludge and the source of lime. Lime stabilization can be used prior to the dewatering of the sludge (called “lime pre-treatment”), or after dewatering (called “lime post-treatment”). In lime-pretreatment, lime is added until the pH is 12 or higher, and maintained at this level for over 2 hours. This will ensure pathogen destruction and prevent the pH of the sludge to drop below 11 for several days. Lime post-treatment involves the addition of quicklime or hydrated lime, but quicklime is preferred since it undergoes an exothermic reaction, which raises the temperature to around $50^{o}C$, which is sufficient to kill the eggs of worms (Metcalf & Eddy, 1991).

Heat Treatment

Heat treatment is a relatively short process (approximately 30 minutes) which involves continuous heating of sludge under pressure. Temperatures reach up to $500^{o}F$ ($260^{o}C$) and pressures reach up to 400 lbf/in^2 gage (2760 kN/m^2) (Metcalf & Eddy, 1991). Heat treatment is a very capital-intensive process and is often only used in plants greater than 5 MGD, or in plants which have limited space. Another associated negative aspect of heat treatment is that the supernatant from this process has an extremely high BOD and often necessitates treatment prior to its return to the head of the plant.

Anaerobic Sludge Digestion

Anaerobic digestion is a process by which organic and inorganic matter is decomposed in the absence of oxygen. In the process of converting 40 to 60 percent of the sludge's organic solids, CO_2, CH_4 and a host of other by-products are created; the remaining organic matter is chemically stable (Franzini *et al.*, 1992). This process is very old, dating back to the 1850's, and is very widely used around the United States, especially in large facilities (that is, facilities treating >5 MGD).

Anaerobic digestion can occur in many different forms. The following discussion will focus on standard-rate digestion, single-stage high-rate digestion, and two-stage digestion.

Standard-rate digestion is a process in which digestion, thickening and supernatant formation occur in the same unit. This process is used mostly in small installations since stratification often occurs, thus rendering half of the tank volume unused.

Single-stage high-rate digestion basically only differs from the standard-rate digestion in that it has better mixing and a higher loading rate.

Two-stage digestion is a process which utilizes two separate tanks. The first tank's purpose is to digest the sludge and heat it (necessitating proper heating equipment). The second tank is used to store and thicken the sludge. The first tank must be enclosed, but the second tank can be open, unheated, or even a sludge lagoon.

Anaerobic digestion tanks are enclosed due to a variety of reasons. The main reasons are to retain heat and odors, to permit the collection of gases (CH_4), and to maintain an anaerobic state (Franzini *et al.*, 1992).

Anaerobic digestion occurs simultaneously in a couple of stages. In the first stage, it involves organic matter being broken down into CO_2, CH_4, H_2S, and organic acids by acid-forming (facultative) bacteria. In the second stage, CH_4 and CO_2 are formed from the organic acids by acid-splitting methane-forming bacteria (strict anaerobes which are very sensitive to their environment) (Hammer & Hammer, 1992). The second stage is a form of gasification, which prevents the pH from dropping. If the second stage doesn't take place, the pH will drop to a level at which digestion would no longer occur. Often, engineers consider anaerobic digestion as a three-stage process. In this case, the first stage is the transformation of higher-molecular-mass compounds into suitable forms for consumption as an energy source (Franzini *et al.*, 1992).

As stated earlier, CH_4 is a by-product of anaerobic sludge digestion due to methanogenesis, which can only take place in the absence of oxygen. Methane forms from the conversion of CO_2 and H_2 into CH_4 and H_2O, or the conversion of acetate to CH_4, CO_2, and H_2O (Franzini *et al.*,1992). This production of CH_4 removes compounds which would prohibit the growth of acid-forming bacteria (acidogens). One of the advantages of using anaerobic sludge digestion is that this CH_4 can be recovered and burned to produce energy. Many facilities around the world use the produced CH_4 to generate power for the entire facility, often a surplus of power, which can be sold to the local electric utility.

A common disadvantage with digesters is that the sludge necessitates heating, a unit which often entails high maintenance. The heating is necessary to ensure that the temperature of the sludge entering the reactor is the same as that of the sludge currently being digested. Digester liquid is often sent to an external heat exchanger to maintain desired temperatures (Franzini *et al.*, 1992). Another drawback of anaerobic sludge digesters is that they are very capital intensive, thus small facilities often resort to aerobic digestion. One more problem associated with sludge digesters is their start-up. Due to the slow growth rate and sensitivity of the acid-splitting methane-forming bacteria, start-up is often slow (Hammer & Hammer, 1992). The number of these microorganisms is very low in influent sludge compared to the amount of acid-forming bacteria, thus compounding the problem. Start-up can take weeks if a seed is used, that is, sludge from a properly operating anaerobic digester. If no seed is available, the start-up period is on the order of several months, which involves careful monitoring and data analysis. Also, the bacteria are sensitive to the toxic metals from industrial wastes, thus hindering or ending digestion.

One form of anaerobic digestion, although rarely used, is thermophilic anaerobic digestion. This is very similar to conventional anaerobic digestion except for the fact that the temperature in the digester is held at a thermophilic level of 120–135^{o}F (49-57^{o}C). This process is based on the fact that biochemical reaction rates, due to microbial activity, double approximately every 10^{o}C (18^{o}F) rise in temperature. This process has the advantage of rapidly digesting sludge and increasing the destruction of bacteria. The disadvantages include the cost to keep the temperature so high, and a low process stability.

It is often the case that in developing countries anaerobic digestion is too costly, with respect to capital and operation and maintenance costs.

Aerobic Sludge Digestion

Aerobic sludge digestion is the stabilization of sludge via long-term aeration in which volatile solids are biologically destroyed (Hammer & Hammer, 1992). This method of stabilization is most applicable to waste activated (biological) sludge or a mixture of waste activated sludge with primary (or CEPT) sludge. Common disposal is land spreading, lagooning, and application to drying beds (disposal is discussed below).

The major advantages of utilizing aerobic digestion over anaerobic digestion include a lower BOD in the supernatant, lower capital costs, ease of operation, and that aerobically digested sludge retains more characteristics and values of a fertilizer than does anaerobically digested sludge. Aerobically digested sludge is often characterized by poor settleability and low solids content, which causes difficulties in disposal of this large volume of sludge. Aerobic digestion and anaerobic digestion are equivalent in terms of their destruction of volatile solids. But, there are disadvantages to aerobic digestion. These include the process sensitivity to temperature, location, and material of tank; another disadvantage is the high power cost to operate the aerators; also, aerobic digestion doesn't allow for the recovery of methane gas for power generation.

The process of aerobic digestion is similar in theory to that of the activated sludge process. Microorganisms proliferate until the food to microorganism ratio becomes too low, thus causing an endogenous condition. This endogenous condition is when microorganisms are forced to metabolize their own protoplasm for food. The protoplasm cannot be replaced since the food available is so scant in this phase of digestion, thus the microorganisms undergo lysis. Here, cells die and their remaining nutrients are utilized by the cells which are still alive (a form of cannibalism, so to speak).

According to Metcalf & Eddy (1991), the most important factors in the design of conventional aerobic digesters include temperature (as microbial biochemical reactions double every 18°F [10°C] rise in temperature), tank volume (hydraulic detention time), oxygen requirements, solids reduction (a major objective of aerobic digestion), process operation (as the pH changes due to the conversion of ammonia to nitrate), and the energy requirement for the mixing/aeration of the sludge.

One form of aerobic digestion is high-purity oxygen aerobic digestion. The modification is implied in the name -- high-purity oxygen is substituted for air. The tanks can either be covered or open. Covered tanks use mechanical mixers to introduce oxygen into the sludge, as the atmosphere above the sludge/liquid surface is high in high-purity oxygen. A covered system also will hold in the heat produced by the exothermic reactions occurring, thus increasing the temperature (which will increase the rate of biochemical reactions). For an open tank system, oxygen is released from the bottom of the tank in fine bubbles so that it is dissolved by the time it reaches the air-liquid sludge interface. Either form of high-purity oxygen aerobic digestion is very costly due to the use of high-purity oxygen in the process.

Thermophilic options exist for aerobic digestion, as they do for anaerobic sludge digestion. The process is similar to that of thermophilic anaerobic digestion, except that the process is now aerobic. It has the same advantages of thermophilic anaerobic digestion (increased pathogen kill, lowered detention times…), yet similar disadvantages (notably its cost for maintaining high temperatures).

It is often the case that in developing countries aerobic digestion is too costly, with respect to capital and operation and maintenance costs.

Composting

Engineers, and other people, have realized that landfill space is decreasing, and that stabilized sludge can be used beneficially as a fertilizer, thus, composting has been utilized and is currently increasing in popularity. In the process of composting, organics stabilize through biological degradation. The process of composting also achieves high temperatures in the pasteurization range (50 to 70°C), which destroys enteric pathogens. The main microorganisms utilized in this process are bacteria, fungi, and actinomycetes (filamentous bacteria for which there are 23 species which thrive in a temperature range of 55-75°C (Brock *et al.*, 1991)).

Composting undergoes three stages: mesophilic, thermophilic, and back to mesophilic (a cooling stage). In the first stage, the temperature of the compost pile increases (via degradation of volatile solids and other biological processes) to about 40°C, in which mesophilic microorganisms prosper. Microbial activity eventually raises the temperature to about 70°C, in which the mesophilic microorganisms no longer thrive and thermophilic microorganisms now thrive. This is the part of composting in which most stabilization and pathogen reduction occurs. As microbial activity slows down, the temperature goes back down (cooling) into the mesophilic range, where mesophilic microorganisms replace the thermophilic microorganisms.

The different methods of composting include the aerated static pile, the windrow method, and the in-vessel composting system. In the aerated static pile method, pipes run below the piles in which air is blown. This air supplies the microorganisms with oxygen, controls the temperature of the pile, and removes excess moisture. The windrow system mixes and turns the piles, to achieve the same goals as the aerated static pile. The in-vessel system achieves stabilization and pathogen destruction inside an enclosed vessel.

It should be noted that before any of these methods can be undertaken, the sludge must first be properly prepared. That is, bulking agents (such as wood chips) and/or an amendment (such as sawdust) to increase porosity, increase air voids, and to provide structural support need to be added (Metcalf & Eddy, 1991).

Sludge Conditioning

The purpose of sludge conditioning is to improve the dewatering characteristics of sludge to make it more manageable. Several methods exist to condition sludge, but the two most commonly used methods are chemical addition and heat treatment.

Chemical addition often involves the addition of polymer to sludge prior to mechanical forms of dewatering. Heat treatment is discussed above under the "Stabilization" section.

Disinfection

Disinfection is often utilized to achieve greater pathogen destruction and bacterial growth-rate control than that achieved in stabilization of sludge alone. Several methods exist, including pasteurization, high pH treatment (such as by lime addition), irradiation, chlorine addition, and the addition of other chemicals.

Dewatering

Dewatering is a mechanical or physical means to reduce the water (moisture) content of sludge, thus decreasing sludge volume. It is done for several purposes, including the following: volume reduction to allow for lower trucking costs in its disposal, moisture content reduction for further processing which requires a higher solids content, and often it is easier to handle when thickened. Several methods are used for dewatering, including vacuum filtration, solid bowl centrifugation, imperforate basket centrifugation, belt filter pressing, recessed plate filter pressing, sludge drying beds, and sludge lagoons (Metcalf & Eddy, 1991). The only ones to be discussed below will be sludge drying beds and sludge lagoons.

Sludge Drying Beds

Sludge drying beds are one of the most widely used methods to dewater sludge. They can be used to dry many different types of sludge, but are most often used to dry digested sludge. The advantages of using sludge drying beds include low capital investment (if land is readily available and inexpensive), little maintenance, low-skill level required by operator, low energy consumption, low sensitivity to variability of sludge, and a yield of a greater solids content. The disadvantages include the requirement of large areas of land, the sludge applied to the beds must be stabilized, the removal of the sludge is highly labor intensive, and the design is very specific to the location's climate (Metcalf & Eddy, 1991).

Although there are four main types of sludge drying beds used (conventional sand, vacuum-assisted, paved, and artificial media), only conventional sand drying beds will be discussed here. Conventional sand drying beds consist of an underdrain system covered by gravel, with a layer of sand over that (9-12 inches deep). The water in the sludge drains by gravity into the drains and evaporates, although drainage accounts for most of the moisture removal. Depending on the location, drying beds that are covered can handle and dry more sludge per year than uncovered beds since the cover system will keep out rain and snow. In the design of these drying beds, a pathway for a truck must be included to allow sludge removal from the beds.

Sludge Lagoons

Sludge drying lagoons are often used in lieu of sludge drying beds. The advantages of using a sludge lagoon is the low energy consumption, further stabilization of organic matter, low capital cost (if land is readily available and inexpensive), and it requires the least amount of skill from an operator. The disadvantages of a sludge lagoon include the potential of odor problems, possible vector problems, and the possibility of polluting groundwater. Moreover, sludge lagoons are land intensive, and sometimes not very aesthetic. The design of sludge lagoons, as with sludge drying beds, is dependant on the local climate since any form of precipitation, and cold temperatures, retard the dewatering process. The residence time in these lagoons is on the order of months to years. Typically, a lagoon is filled for a specific period of time (such as 18 months), then is left alone for about one half of a year (Metcalf & Eddy, 1991).

Land Application

The land application of sludge has been practiced for several years in the US (Hammer & Hammer, 1992). Depending on regulations, sludge must be stabilized before it is applied to land. It is also important to understand the characteristics of the sludge, such as its nutrient content, organic content, pathogen content, proximity of soil surface to the water table, and its metals content. This will determine its applicability to varying soils and what is grown on that land.

Sludge Disposal

Other than land application, sludge can be landfilled or lagooned. The landfilling of sludge is often appropriate (according to regulations) for stabilized and dewatered sludge, and incinerated sludge. Lagooning is similar to lagoon drying beds. Lagoons are filled with sludge and the sludge further stabilizes and the water on top is decanted.

Sludge Quantity

The quantity of sludge produced is what will determine the size of sludge handling facilities and often the choice of processes for treating the sludge. The quantity of sludge produced by conventional primary treatment is equal to the quantity of total suspended solids removed by the treatment process. Thus,

$$S_p = Q \cdot TSS_{rem} \cdot 10^{-3} \tag{6-1}$$

where:

S_p = Dry weight of raw sludge produced (kg/d)

Q = Influent flow rate (m^3/d)

TSS_{rem} = Concentration of suspended solids settled/removed (mg/L)

10^{-3} = Conversion factor for liters to m^3 $(\frac{1000L}{1m^3})$ and mg to kg $(\frac{1kg}{10^6 mg})$

This, of course, is in terms of dry solids. Therefore, this value must be turned into a volume by using the sludge %solids content and sludge density.

CEPT sludge calculations must include the chemicals which precipitate out ($Fe(OH)_3$) and the precipitates formed in the removal of phosphorus. The equation, from Murcott and Harleman (1992), to include these variables, is:

$$S_p = Q \cdot [TSS_{rem} + F \cdot P_{rem} + K \cdot C_c] \cdot 10^{-3} \qquad \textbf{(6-2)}$$

where:

F = Stoicheometric factor for $FePO_4$ removal (1.42 for $FeCl_3$, a trivalent metallic salt)

P_{rem} = Quantity of phosphorus removed (mg/L)

K = Constant (0.66 for $FeCl_3$; 66% by weight of the $FeCl_3$ precipitates out as $Fe(OH)_3$),

C_c = Concentration of chemical coagulant added (mg/L)

For Alternative 2, the sludge produced is determined by:

Q = 13911 m^3/d

TSS_{rem} = 180 mg/L

F = 1.42

P_{rem} = 3 mg/L

K = 0.66

C_c = 50 mg/L of $FeCl_3$ (produced by NHEEL)

%Solids = 4%

Sludge Density = 1025 kg/m^3.

Thus,

S_p = 3022 kg/d

Wet Sludge = $\frac{S_p}{\%solids} = 75{,}559$ kg/d

Sludge Volume = $\frac{S_p}{(\%solids)(\rho)} = \frac{WetSludge}{\rho}$ 74.1 m^3/d.

This is the amount of sludge that will necessitate handling on a daily basis[28]. Thus, the sludge handling facilities (discussed in Chapter 6) will be designed to handle this volume.

For the sludge production of Alternative 3, the difference from Alternative 2 is that C_c = 30 mg/L. Following the same calculations as done above for Alternative 2, the above results are:

S_p = 2838 kg/d

Wet Sludge = 70,698 kg/d

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[28] It should be noted that the calculations for sludge accumulation in the in-pond CEPT lagoon and the sedimentation basins, C_c is not adjusted for the fact that chemical dosing is not a 24-hr process. Thus these sludge calculations represent a conservative figure.

Sludge Volume = 69.58 m^3/d

This sludge volume, as stated, is the sludge produced daily. But, the sludge will be allowed to accumulate in the in-pond CEPT lagoon for two years. Thus, due to compaction and anaerobic digestion, the volume and amount of solids after two years cannot simply be determined by multiplying the above results by 2 years. The amount of sludge accumulation is calculated as follows:

The sludge in the lagoon is assumed to degrade at a rate of 50% of volatile solids (VS), which represent 75% of the total solids (TS), per year. The remaining solids will be called non-volatile solids (NVS), which are also often called Fixed Solids (FS). The following calculations will determine the TS, VS and NVS accumulation per year and then determine the amount of dry solids after 2 years:

$TS = S_p \cdot 365$ d/yr = 1,036,137 kg/yr

$VS = TS \cdot 75\%$ = 777,103 kg/yr

$NVS = TS \cdot 25\%$ = 259,034 kg/yr

$$\text{Dry solids (kg)} = 2 \cdot (NVS) + (0.5)^2 \cdot (VS) + 0.5 \cdot (VS) \qquad \textbf{(6-3)}$$

$$= 1100896 \text{ kg/2yr}$$

Thus, 53.2% ($\frac{1{,}100{,}896}{(2)(1{,}036{,}137)}$) remains from what was produced over a two year period. The result is that after two years of using the in-pond CEPT lagoon, there will be 1,100,896 kg of dry solids. Assume, for a moment, it occupies its original volume (that is, no compaction), this would yield a %solids of 2.125%. But, the sludge is compressed via sludge compaction (sludge settling on top of sludge). Assuming that compaction results in a decrease in sludge height of 65% (thus, a height of 35% compared to no compaction), the resulting density of sludge is 60 kg of dry solids per m^3 of wet sludge (6% solids). The %solids is calculated as follows:

$$= \frac{Solids\ Pr\ oducedPerDay}{VolumeOfSludgePerDay \cdot (1 - \%compaction)}$$

$$= \frac{151.9 \frac{kg}{d} DrySolids}{(69.58 \frac{m^3}{d} WetSludge) \cdot (1 - 0.65)}$$

≈ 6 % solids

Thus, the new density (of the wet sludge) is

(6%)(1.5 g/cm^3) + (94%)(1.0 g/cm^3) = 1030 kg/m^3.

Above, 1.5 g/cm^3 is the density of dry solids, and 1 g/cm^3 is the density of water.

With this calculated density (of the wet sludge), we can calculate the volume of wet sludge produced after two years, and the weight of wet sludge.

Wet sludge weight after two years $= \dfrac{DrySolids(2yrs)}{\%solids}$

$$= \frac{1,100,896kg}{6\%}$$

=18,348,266 kg (after 2 yrs)

The wet sludge volume is the wet weight divided by the wet density (1030 kg/m^3). Thus, the volume is

$$\text{Volume} = \frac{18{,}348{,}266 kg}{1030 kg/m^3}$$

$$= \frac{17814 m^3}{2 yr} \text{ [that is, after 2 years residence time]}$$

$$= 24.4 \text{ m}^3/\text{d}.$$

Thus, dry solids production per day, including decomposition and compaction is

$$= (24.4 m^3/d)(1030 kg/m^3)(6\% solids)$$

= 1510 kg dry solids per day.

The calculations above will determine the number of drying beds necessary. The calculations for the number of sludge drying beds are in the Design Chapter (CH. 7).

Chosen Sludge Handling Methods

As is evident, many different methods can be utilized to process sludge. The main difference in most approaches is the cost.

The options to be discussed below were chosen based on technical feasibility (often simplicity) and a cost-benefit analysis. Many processes were eliminated due to high cost, such as anaerobic and aerobic sludge digestion. Also, the options below require very little technical expertise and very little maintenance (as opposed to many of the aforementioned processes involving many mechanical parts and sophisticated design criteria). Land is readily available at the Tatui site, thus the facility was designed with this important characteristic (*i.e.*, available land).

Alternative 1 (Aerated Lagoon System)

SABESP's design calls for the sludge to accumulate in the settling lagoons and have a residence time of two years. After this two-year residence time, where the sludge will anaerobically digest, the sludge will be pumped (by a barge) into sludge drying beds (conventional sand drying beds).

Alternative 2 (CEPT as Pre-Treatment for Lagoons)

For Alternative 2, sludge will be pumped from the CEPT sedimentation basins to a filter press. The sludge will be properly mixed with bulking agents and/or an amendment for which it will be composted in a windrow composting facility (see Chapters 6 and 7 for design information on the windrow composting facility). The compost will be disposed of by landfilling, or given away (free) as fertilizer (for example, to be applied to eucalyptus trees, which are widely used in Brazil as a raw material for "pressed" bedding boards).

Alternative 3 (In-pond Chemical Precipitation)

The third alternative/design for Tatui does not call for the use of chemically enhanced sedimentation basins. The design involves a "chemically enhanced lagoon" which is located where the current anaerobic lagoon is (but does not take up the whole lagoon). Thus, the sludge will settle in the lagoon and accumulate at the bottom of the lagoon. After a period of two years (a time in which the sludge volume has become stable and volatile solids are reduced by a rate of 50% per year due to anaerobic microbial activity in the sludge), the sludge will be pumped from the bottom of the lagoon, by a barge, into sludge drying beds at the rate at which the sludge is being produced (see design specifications in Chapter 7). The residence time in the sludge drying beds is 24 months. The sludge will be mechanically removed after its 24-month residence time and will be subsequently landfilled, or given away as free fertilizer, if possible.

Chapter 7 - Design

Introduction

This chapter will discuss and explain how wastewater facilities are designed, focusing mainly on Alternative 2 and Alternative 3 for Tatui with minimal design analysis of Alternative 1.

The design, and selection, of a wastewater treatment facility is based on the study of the following items: the physical, biological and chemical characteristics of the wastewater; the applicable discharge standards which must be met; and the environment to receive the effluent and to what level it must be maintained/protected (Franzini *et al.*, 1992). Each item was addressed in the design of Alternatives 2 and 3. The first item was taken care of, to the best degree possible, by the Tatui field study and jar tests. The applicable standards which must be met in Brazil are a maximum effluent BOD_5 of 60 mg/L. Alternatives 1-3 all meet this standard, therefore this item has been addressed. Presumably, in setting this standard, Brazilian regulations have addressed the last item – the environment to receive the effluent and the level to be maintained and protected.

Bar screens, grit chambers, sedimentation basins, pre-pond precipitation treatment systems, and in-pond CEPT lagoons are discussed below. In addition, chemical pumping and storage units, and sludge handling facilities are presented. Sludge processing facilities are discussed in Chapter 6, whereas other processes, such as activated sludge, will not be discussed due to their exclusion from the Tatui designs.

Bar Screening

Bar screens are utilized in wastewater treatment facilities for the purpose of removing coarse solids. These coarse solids are mostly latex (condoms), sticks, plastics, and rags. More generally stated, bar screens are for the purpose of removing any large object which may enter a sewerage system. Often, bar screens stop money (paper), jewelry, shoes, and it has been rumored that a bar screen once stopped a motorcycle from entering a treatment facility. The most common bar screen involves metal bars which are pre-designed to have a specified width and spacing, to facilitate the removal of the coarse solids. The bar spacing is typically 1 to 2 inches, inclined at an angle of 30 to 75° to the horizontal for manually cleaned bar screens. For mechanically cleaned bar screens, the bar spacing is often ½ to 1½ inches, inclined at an angle of 45 to 90° to the horizontal (Reynolds *et al.*, 1996). A benefit of a mechanically cleaned bar screen is that with the smaller spacing, smaller particles can be removed than with a manually cleaned bar screen. Another advantage is that the screens can be cleaned 24 hours a day and do not depend on a worker being there to clean the screens. A disadvantage of mechanical bar screens is the maintenance problems associated with them.

The channel that conveys the wastewater influent to the bar screens should be straight for several feet before reaching the bar screens. The channel should be properly sized to maintain a velocity of at least 1.25 ft/s (.4 m/s) to prevent grit deposition in the conduit. The flow-through velocity should be as close to 2 ft/s (.62 m/s) as possible, with a maximum flow-through velocity of 3 ft/s (0.9 m/s) to avoid pass through (Metcalf & Eddy, 1991).

The head loss through mechanical bar screens is usually kept below 6 inches. The mechanical rake begins to clean the bar screens when a specified pressure drop is detected, thus maintaining a head loss below 6 inches. The following equation from Reynolds *et al.* (1996) is used to calculate the head loss through bar screens:

$$h_L = \frac{(V_b^2 - V_a^2)}{2g} \cdot \frac{1}{0.7} \quad \textbf{(7-1)}$$

where

h_L = head loss, ft (m)

V_a = approach velocity, ft/sec (m/s)

V_b = flow through velocity, ft/sec (m/s)

g = gravitational acceleration (32.2 ft/s^2, 9.81 m/s^2)

The bar screens for Tatui were designed by SABESP[29]. The specifications for design state that the width of each bar is 3/8 inches, with a thickness of 1½ inches and a bar spacing of ¾ inches. This (manual) bar screening facility is a combined bar screen/grit chamber. That is, the head of the grit chamber is a bar screen. This unit will be the same one used in Alternatives 2 and 3. The unit has been analyzed with scrutiny and semblance to common design practice and is deemed appropriate for Alternatives 2 and 3.

Below are pertinent calculations. The cross-sectional areas and other pertinent design criteria were obtained from SABESP's blue prints for Alternative 1.

Approach velocity in the inlet conduit:

---

[29] Often the wastewater passing through bar screens is subsequently comminuted. This means that the wastewater passes through grinders which grind the coarse solids. This is often done to prevent damage to pumps and other processes downstream. This will not be used in any design alternative in this report.

$Q = VA$.

Where,

Q = flow (m^3/s)

V = velocity (m/s)

A = Cross-sectional area (m^2)

Thus,

$$V = \frac{Q}{A} = \frac{.16101 m^3/s}{\frac{\pi (0.7m)^2}{4}} = .4184 m/s$$

Approach velocity in the bar-screen/grit-chamber leading up to the bars:

$$V_a = \frac{Q}{A} = \frac{.16101 m^3/s}{(0.7m)(.9811m)} = .234 m/s$$

Flow-through velocity:

The flow-through velocity is calculated by determining the flow-through area taking into account the width of the bars and the spacing between the bars (of which there are 35). The continuity equation (Q=VA) is again used.

$$V_b = \frac{Q}{A} = \frac{.16101 m^3/s}{(.7m)(.6477m)} = 0.36 m/s$$

Head loss through the bar screens:

$$h_L = \frac{[(.36 m/s)^2 - (.234 m/s)^2]}{2(9.81 m/s^2)} \cdot \frac{1}{0.7} = 0.2 in$$

The head loss is below 6 inches, which is satisfactory. The approach is satisfactory; the flow-through velocity is a bit low, but is satisfactory. Since the calculations were done by SABESP, it is not known whether or not this unit is designed under peak or average conditions. The above calculations are done using average flow; if peak flow is used as the design parameter, $V_b = 0.7$ m/s, which is also satisfactory (and not low, but still under the upper limit of 0.9 m/s).

The screenings (that which is removed by the bar screens) will be landfilled or handled according to the governing legal code; this will be the handling mode for the alternatives.

Grit Chambers

Upon passing through bar screens and/or a comminutor, the next unit process is a grit chamber. Grit includes eggshells, coffee grounds, bone chips, and sand (the specific gravity of the removed particles ranges from 1.6 to 2.65) (Franzini *et al*., 1992). Grit is often defined as sand particles of 0.2 mm in diameter or greater (Heinke *et al*., 1989). The purpose of a grit chamber is to remove these coarse particles that can interfere with and/or damage pumps, and to prevent the accumulation of grit in other unit processes and sludge digesters (Franzini *et al*., 1992). The velocity in the grit chamber must be kept high enough to prevent organics from settling, but must be low enough to allow coarse particles to settle.

Sedimentation Basins

Sedimentation basins (often referred to as sedimentation tanks or primary clarifiers) are, most often, the unit process after a grit chamber. Sedimentation is the separation of solids from wastewater through settling. The purpose of sedimentation basins is the removal of suspended solids and their associated BOD. The particles that settle are the ones that have a higher specific gravity than that of water. The main parameters for the design of a sedimentation basin include the surface overflow rate and the detention time.

The surface overflow rate (OFR) is defined as the volume of wastewater divided by the surface area of the basin (in units of length per time). That is, $OFR = \frac{Q}{SA}$.

The range of overflow rates for conventional sedimentation basins is 800 – 1200 gal/ft^{2}d (32.6 – 48.9 m/d). The OFR value is also the basis for design of a chemically enhanced primary sedimentation tank (as shown below in the calculation for sizing the sedimentation basins).

Detention time is the amount of time that wastewater spends in the basin. Conventional sedimentation basins are designed to have detention times varying between 1.5 hours and 2.5 hours (typically, 2 hours). Chemically enhanced primary sedimentation basins can be designed at much shorter detention times due to increased settling velocities from the addition of chemical coagulants. This allows for the option of treating more wastewater in the same amount of time compared to a conventional tank. Or, the tank can be sized smaller, but still treat the same amount of wastewater that a conventional basin can.

Settling occurs mostly through physical processes. The most common physical process for removing the TSS in wastewater is gravity settling. This gravitational settling is employed in several places: in the grit chamber to remove grit, clarifying raw sewage and concentrating the settled solids (primary sludge), clarifying and concentrating biological suspensions (activated or secondary sludge), and in gravity thickening of sludges (Heinke *et al.*, 1989). Settling is often modeled as "ideal settling" in which particles settle at a constant settling velocity V_c (from equation 3-1). It should be noted that due to factors such as turbulence (particularly encountered at tank inlets and outlets), short circuiting of flow, dead spots (no flow) in the tank, and the movement of sludge collectors (be they mechanical or manual), ideal settling will not occur (Heinke *et al*., 1989). To suppress short circuiting in Alternatives 2 and 3, baffles will be used.

The basis of design of the sedimentation lagoons for Alternative 1 is based on SABESP's assumptions and undisclosed calculations, and thus will not be covered here.

Basin Design

The basis for the design of the sedimentation basins for Alternative 2 is the settling test. As can be seen by the NHEEL (a $FeCl_3$ producer in Brazil) settling tests (Figures 7-1 through 7-3), the necessary time to settle in the jar for optimal COD and TSS removal was 3 minutes. This settling test will determine the OFR for the actual basins. To correlate this with an actual sedimentation basin, the following equation must be used:

$$\mathrm{OFR} = \frac{H}{t_d}$$

Where,

OFR = Overflow rate (m/d)

H = height of water in jar test (m)

t_d = necessary settling time (d).

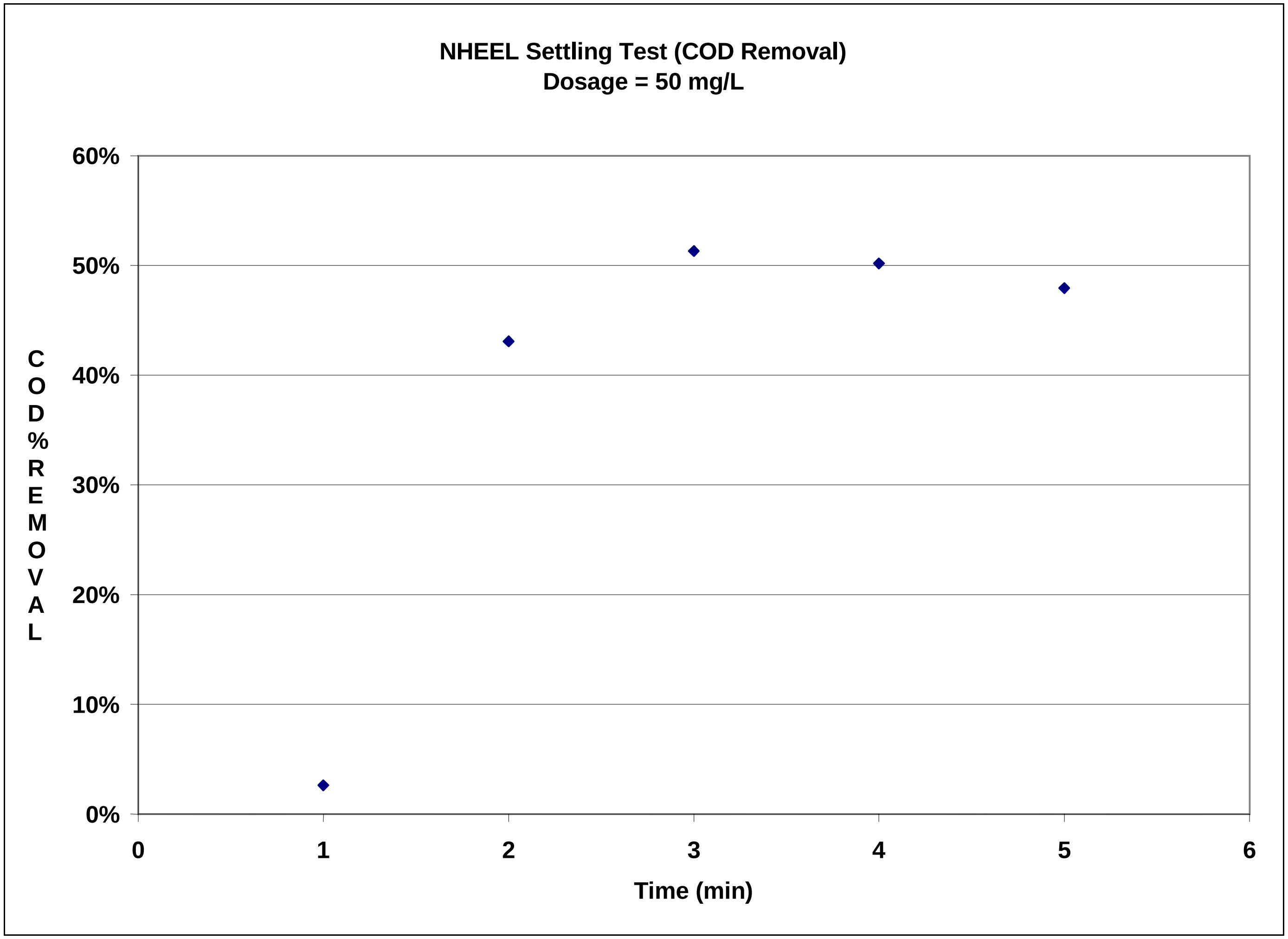

Figure 7-1: NHEEL Settling Test Results (COD)

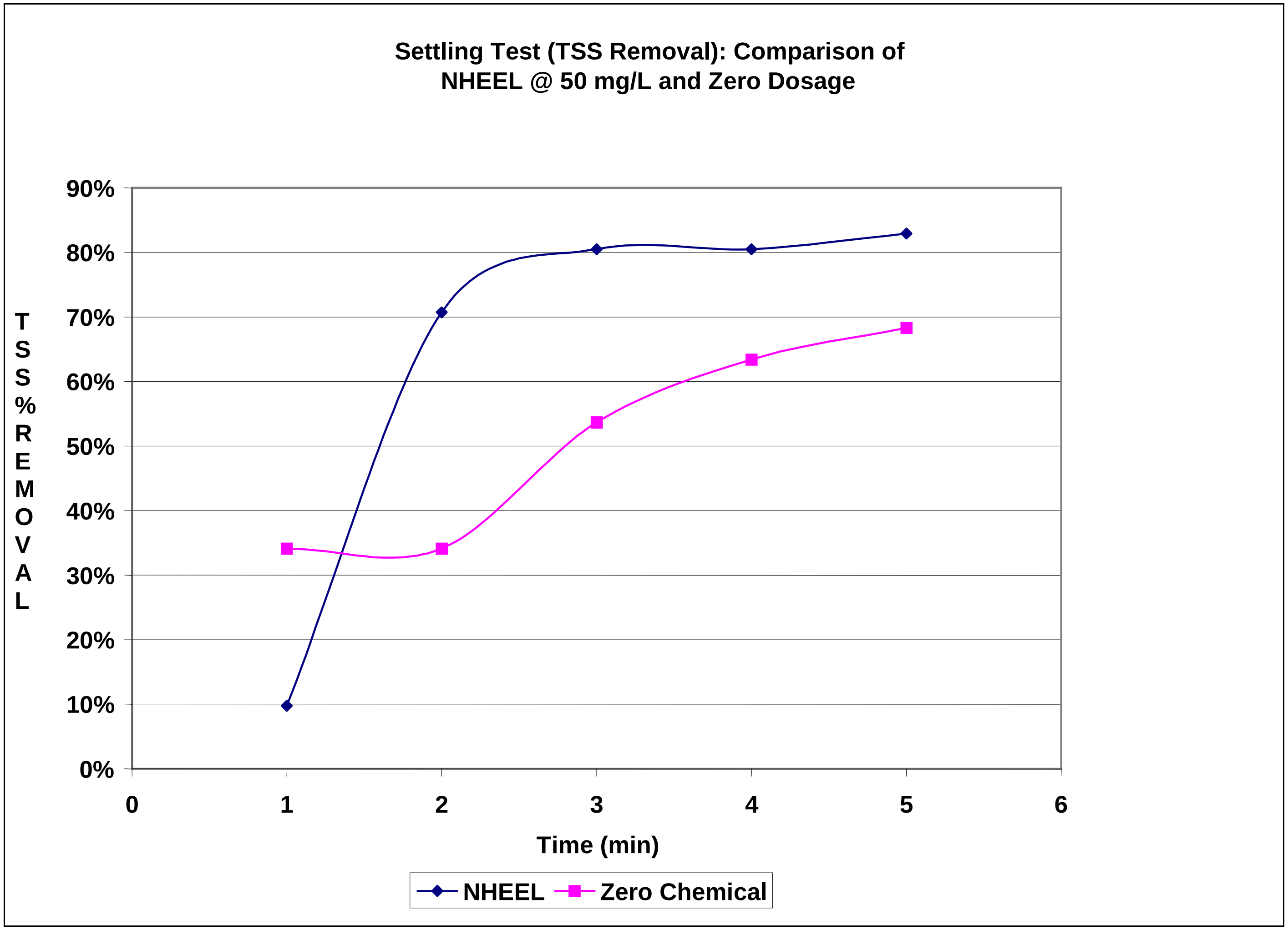

Figure 7-2: NHEEL v Zero Chemical Settling Test Results (TSS)

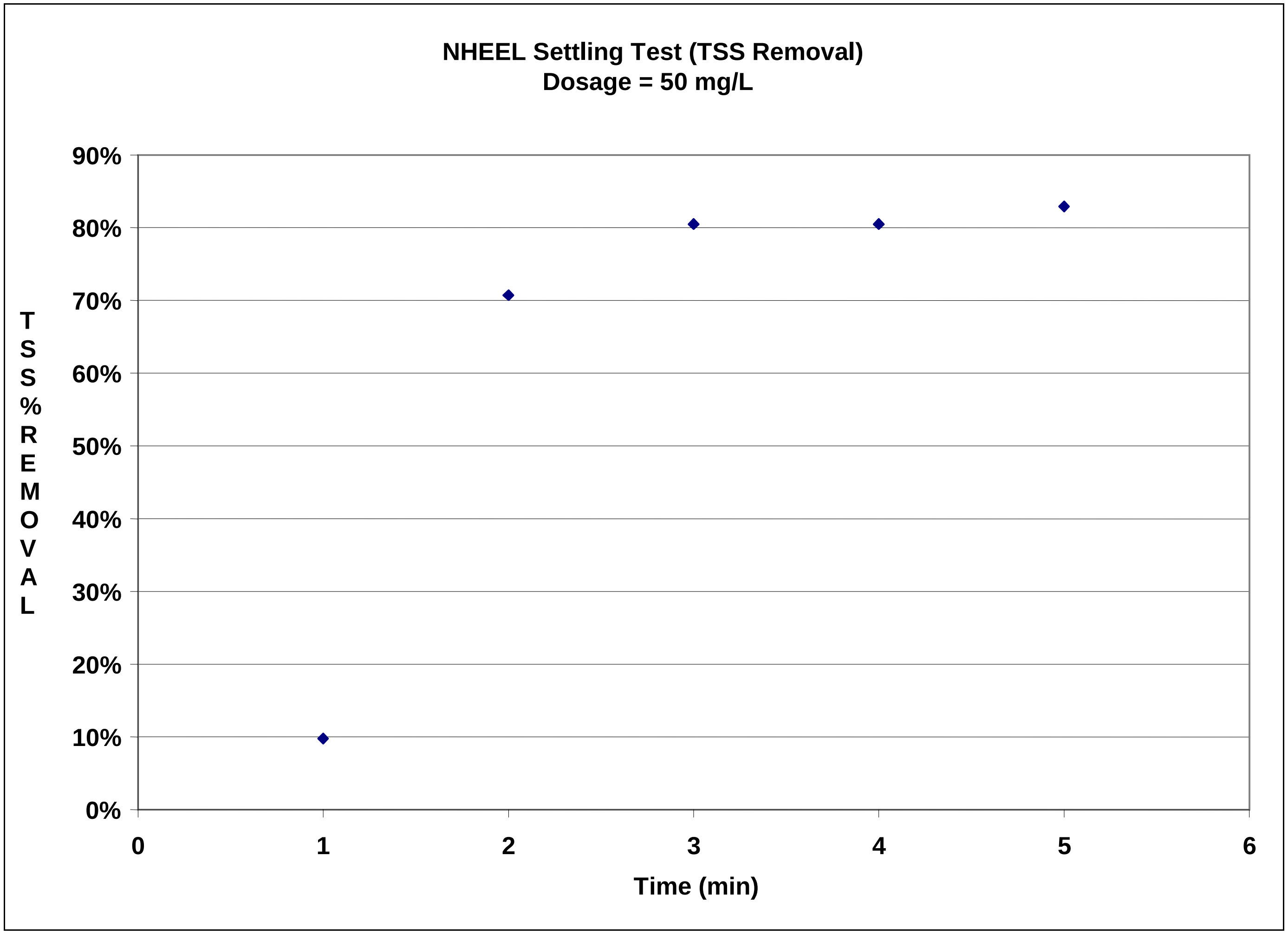

Figure 7-3: NHEEL Settling Test Results (TSS)

The height of water in the jar test beaker was 6 inches (0.1524m) with a 2-liter sample volume. Thus,

$$\text{OFR} = \frac{.1524m}{3\min \cdot \frac{1hr}{60\min} \cdot \frac{1d}{24hr}} = 73 \text{ m/d.}$$

With this value (73 m/d), the primary sedimentation basins were designed as follows:

> Sedimentation basins are designed for average flow, thus the design value for flow is 161.01 L/s (with a BOD_5 of 276 mg/L), as specified by SABESP. Another design parameter is that the length to width ratio should be at least 5:1 to ensure horizontal flow [the same ratio applied to L:H]. To be able to handle the flow without having over-sized basins, and to allow sufficient capacity to handle flow if one basin has to go down for repair, more than one basin is necessary. For Alternative 2, three basins were chosen as sufficient. For Alternative 3, a lagoon is used instead of a basin. These kind of simple lagoons do not have down time for mechanical failure due to the fact that there are no mechanical parts in the lagoon.

Chemical Dosage (C_c) = 50 mg/L

Chemical Dosing Period = 12 hr/d

Q = 161 L/s = 13911 m^3/d

Basin Height (H) = 3.5 m

Basin Width (W) = 3.5 m

Basin Length (L) = 19 m

Surface Area (SA) = W·L = 66.5 m

Volume (per basin) (V) = 232.75 m^3

Total Volume (total of all three basins) (V_T) = 698.25 m^3

Overflow Rate (OFR) = $\frac{Q}{3 \cdot SA}$ (a factor of 3 to account for each basin)

$$= 69.73 \text{ m/d} \quad (\text{less than design value, } \therefore \text{ OK})$$

$$\text{Detention time} \quad (t_d) \quad = \frac{V_T}{Q}$$

$$= 1.20 \text{ hrs.}$$

The peak flow is 224.33 L/s. With this flow, the OFR is 97.15 m/d with a detention time of 0.86 hours.

The sludge produced in each sedimentation basin will be manually raked into the basin's sludge hopper for pumping to the filter press. The plan and section views of the sedimentation basins are in Figures 7-4 & 7-5.

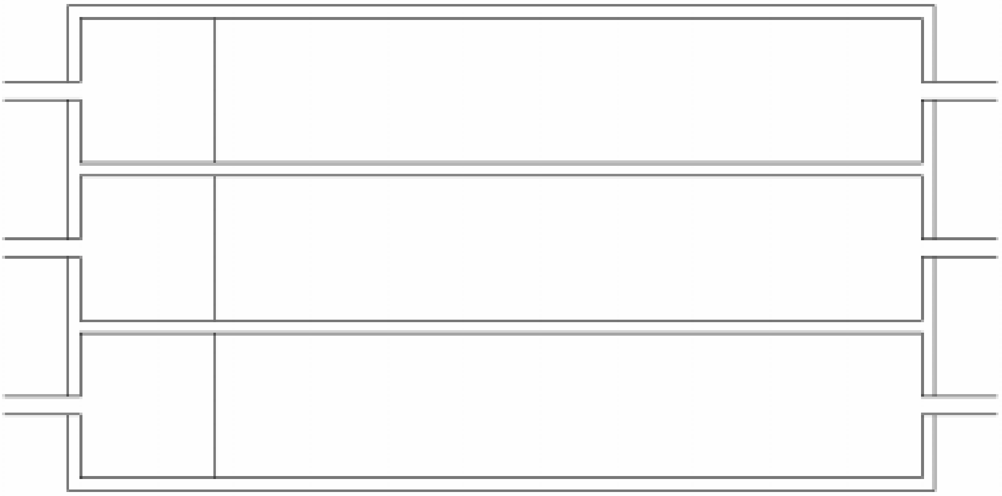

Figure 7-4: Plan View of Alternative 2 Sedimentation Basins

Figure 7-5: Section View of Alternative 2 Sedimentation Basins

In-Pond & Pre-Pond CEPT Treatment Systems

As early as the 1960's, an attempt was made to treat the wastewater in stabilization lagoons by chemical addition (Hanaeus, 1991). These chemically enhanced lagoons work in relatively the same manner as conventional lagoons. The only difference is the chemical addition and thus a higher sludge accumulation due to a higher removal efficiency. As simple lagoons are dependent on solar radiation for their performance, so too is the operation of chemically enhanced stabilization lagoons, although not as heavily dependent.

The objective of lagoons, and all wastewater treatment systems, are the removal of TSS, BOD, and a host of contaminants which can pose a threat to the environment. The removal of BOD is accomplished partially through the removal of organic carbon. The major pathways for its removal includes the separation of particulate organic matter into the bottom of the lagoon's sludge bed or by the degradation of organic matter into CO_2, or during anaerobic conditions, turning the organic carbon into CO_2 and CH_4 (Hanaeus, 1991). Algae, which play a major role in simple lagoons (through the supply of oxygen), can utilize this CO_2 produced in the degradation of organic carbon. But in chemically enhanced lagoons it is chemical precipitation which is the governing treatment process, not algae (Hanaeus, 1991). From a study conducted by Hanaeus (1991) of Finnish CEPT lagoons, it was discovered that the lagoons achieved a BOD_7 removal efficiency of 77%, with a range of 43-88%. These same plants achieved an average phosphorus removal efficiency of 60%, with a range of 20-100%. As in simple stabilization lagoons, short-circuiting of lagoons is detrimental. Thus, it is this hydraulic factor which is the greatest deterrent to the successful operation of chemically enhanced lagoons. To combat this, baffles will be placed in the in-pond CEPT lagoon of Alternative 2 in an attempt to prevent short-circuiting.

In-pond CEPT lagoons are most prevalent and have been most-extensively studied in Scandinavia (where they are called fellingsdams). There are three types of fellingsdams (Balmer *et al*., 1987). The first is pre-pond precipitation. This is what Alternative 2 is, the addition of chemicals in a tank separate from, and before, the lagoon system. The second type is in-pond precipitation. This is what Alternative 3 is, the addition of chemicals at the head of the lagoon system. The third type of fellingsdam is post-pond precipitation. As is indicated by its name, it is the addition of chemicals to lagoon effluent in a separate tank. A study from Balmer (1987) of 56 Norwegian treatment plants (fellingsdams) found that the average removal of TSS was 87.6%. The average removal of BOD was 83.0%, a removal efficiency of COD of 76.9%, and an average removal of TP was 91.6%. It should be noted that not all of these were in-pond precipitation facilities. Of these, that is, the in-pond precipitation facilities, the removal of COD ranged from 68-83%, the removal of TP ranged from 70-94%, and the removal of TSS ranged from 80-93%. Balmer *et al*. (1987) conclude their paper by stating that the simplest and cheapest fellingsdam is the in-pond precipitation mode, which gives the satisfactory results of 70-90% BOD-reduction and 85-95% TP-reduction.

In the planning and operation of fellingsdams, Balmer *et al*. (1987) suggest addressing three matters. The first matter is the need for operator attendance. Alternative 2 is designed to have one engineer, one operations worker, and two maintenance workers (all on-site). Alternative 3 (and Alternative 1) is designed to have an on-site engineer and an on-site operations worker. The second matter is sludge production. For Alternative 2, the sludge production is calculated in Chapter 6, and the sludge will be removed from the sedimentation basins throughout the work-day of the facility's staff. For Alternative 3, sludge production has also been calculated in Chapter 6 and the in-pond CEPT lagoon is sized to have a sufficient detention time even when the sludge has accumulated for 2 years. For Alternative 1, SABESP also has made undisclosed calculations and maintains that its settling lagoons are sized to accumulate sludge for 2 years and still operate efficiently. The third matter is the possibility of odors. Although a normal lagoon will produce odors, CEAGESP is not close to residences, so this is not a significant concern in the design. In the current 2-lagoon system of CEAGESP, the odors were not strong at all, and the visit was in the summer time, when temperatures are high and microbial activity is also high, and still no odor problems.

The in-pond CEPT lagoon will function as an extremely large (earthen) sedimentation basin which is sized properly to store the accumulated settled solids (sludge) for a period of two years. The facility staff will maintain the lagoon; most importantly, as algae form in the lagoon, they will be removed and disposed of. The sludge will accumulate at the bottom of the lagoon and will anaerobically digest. Lagoons become anaerobic due to microbiological activity and the BOD entering the lagoon. The influent BOD is 276 mg/L, and since approximately 50% (based on jar tests) will be removed through chemical coagulation and sedimentation, the concentration to the sludge on the bottom will be about 140 mg/L. This concentration is sufficient over the specified surface area (10,000 m^2) to create an anaerobic condition. Also, microbial activity doubles with a rise of (approximately) 10ºC, and Tatui is in a warm climate (with an average temperature of approximately 22ºC). Thus, with the warm temperature and high rate of microbial activity, dissolved oxygen will be depleted and an anaerobic state in the in-pond CEPT lagoon will be achieved rather quickly. In fact, Narasiah *et al*. (1990) indicate that in high water temperatures in ponds (greater than 18˚C [64˚F]), organic sludge may be completely decomposed. The biodegradation of the accumulated bottom sludge is of great importance in a high-water temperature setting as Tatui, when it may be more rapid than solid decomposition (Hanaeus, 1987). The sludge will digest similarly to sludge in an anaerobic digester, and also in the same way sludge digests in a lagoon without chemical addition. And since the temperature is fairly high, the sludge will "self-digest" quicker than in the studied facilities in Scandinavia.

The in-pond CEPT lagoon will be in the same spot the current anaerobic lagoon is located (see Figure 7-6 for the panorama of the current anaerobic lagoon). But, the current design calls for a surface area of one hectare (10,000 m^2), whereas the current anaerobic lagoon is 2.5 hectares. Thus, an earthen dike will be placed in the current anaerobic lagoon at a specified distance to achieve a 1-hectare surface area for the in-pond CEPT lagoon. The following is the design specifications for the in-pond CEPT lagoon:

Chemical Dosage (C_c) = 30 mg/L of NHEEL ($FeCl_3$).

Depth (H) = 4 m

Volume (V) = 22190 m^3 (assuming no side-slope)

Surface Area (SA) = 10,000 m^2

Overflow Rate (OFR) = 1.4 m/d

Residence Time (t_d)[30] = 38 hr

Chemical Dosing Period = 12 hr/d

Figure 7-6: Panorama of the CEAGESP Anaerobic Lagoon

Storage and Pump Calculations

The chemical dosing for Alternatives 2 and 3 will be handled by a pump and storage facility. The following are the pump and storage calculations for each design for the use of NHEEL, the chosen chemical coagulant.

---

[30] This is the detention time after two years of sludge accumulation has occurred.

Pump Calculations

Alternative 2

Amount of dry chemical required:

$= Q_{max} \cdot C_C$

$= (224 L/s)(50 mg/L)(86{,}400 s/d)(\frac{1kg}{10^6 mg}) = 970 kg/d$ (dry basis)

(dry weight = 970 kg/d) = (wet weight = ?) x (% in solution = 38%)

wet weight = $\frac{970 kg/d}{0.38} = 2550 kg/d$ (liquid solution).

Pump Calculations:

$Q_{max} = 224$ L/s = .224 m^3/s

%Solids = 38%

Density (ρ) = 1.4 kg/L

C_c dry = 50 mg/L

C_c liquid = $\frac{C_C dry}{\%Solids} = \frac{50 mg/L}{.38} = 132 mg/L$

Mass Flux = Q ·C = (806.4 m^3/hr)(.132 kg/m^3) = 107 kg/hr

Pump Capacity = $\frac{MassFlux}{\rho} = \frac{107 kg/hr}{1.4 kg/L} = 76.5 L/hr$ = 500 gpd

Thus, 500 gpd (gallons per day) is the necessary pump capacity.

Alternative 3

For Alternative 3, the same calculations pertain, with the substitution of Cc = 30 mg/L, instead of 50 mg/L. This yields a requirement of 1530 kg/d of liquid solution, and a necessary pump capacity of 300 gpd.

Chemical Storage Tanks

The storage facility for the chemicals will have a volume designed to store two days of peak flow demand.

Alternative 2

Volume = (Pump Capacity)(2 days)

= (500gpd)(2d) = 1000 gallons = 3785 L

Thus, a storage tank of 4000 L will be used, which will provide necessary freeboard.

Alternative 3

Volume = (300 gpd)(2d) = 600 gallons = 2270 L

Thus, a storage tank of 2500 L will be used, which will also provide necessary freeboard.

The chemical storage tanks will be filled as deemed necessary by the plant engineer, most likely, every two to three days.

Existing Design

The existing facility (CEAGESP) at Tatui is a very simple lagoon treatment system. To make it even simpler to operate, the facility had no provision for sludge processing. Thus, the handling it will receive is its removal and subsequent landfill disposal as the lagoons are dredged for the new construction to begin.

Windrow Composting (Alternative 2)

The chosen sludge-handling mode for Alternative 2 is Windrow Composting. The windrow method will be done by mixing and turning the piles to supply the microorganisms with oxygen, to control the temperature of the piles, and to remove excess moisture. Before the piles can be formed, the sludge will be thickened in a filter press. Then amendment and/or bulking agents will be added to the dewatered sludge. The amount of amendment and/or bulking agents can not be presented here since there has not been an analysis of the (dewatered) sludge to determine the optimum mixture. Thus, this is a conceptual design of what Alternative 2 uses to treat its produced sludge. The windrows should be mixed at a ratio of approximately 3:1 with wood chips[31], and formed into windrows of about 8 ft high and 12 feet wide, with a spacing on the order of 8 feet between the piles.

Aerobic Digestion

In the early stages of the project, it was thought that aerobic digestion would be the option to stabilize the chemical sludge produced in the chemically enhanced sedimentation basins of Alternative 2. Calculations showed it to not be a feasible option. To see the calculations, see Appendix B.

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[31] Source: Conversation with Michael Bryan-Brown of Green Mountain Technologies, Whitingham, VT (May 3rd & 6th, 1999).

Sludge Drying Beds

Alternative 1

Since SABESP designed this alternative and the calculations are undisclosed, the following is only the sizing and number of drying beds:

Bed Length = 25 m

Bed Width = 5 m

Number of Beds = 32

Land Requirement = 4000 m^2.

Alternative 3

Alternative 3 will dry the in-pond anaerobically-digested sludge in sludge drying beds. The calculations are:

> From the sludge accumulation calculations and digestion calculations for the in-pond CEPT lagoon (in Chapter 6), the loading of dry solids is 1508 kg/d with a density (ρ) of 1030 kg/m^3 at a 6% solids content. This daily dry solids load amounts to 1,100,913 kg after two years (the designed residence time). The sludge will be pumped out of the in-pond CEPT lagoon at the rate at which it accumulates (1508 kg/d dry solids), which is 24.4 m^3/d (from the calculations presented in Chapter 6). From Metcalf & Eddy (1991) design recommendations, the chosen loading rate is 125 kg/m^2·yr. The drying beds are designed to have a residence time of 2 years. Thus, the loading rate is 250 kg/m^2·2yrs. The required surface area (SA) is calculated by dividing the dry load by the loading rate (for the two-year period), as so:

$$SA = \frac{TotalDrySolids}{SolidsLoadingRate} = \frac{1{,}100{,}913kg}{250kg / m^2} = 4400m^2$$

SABESP's design calls for the use of 25 x 5 m-drying beds. Alternative 3 will use the same (SABESP-approved) dimensions. This surface area will necessitate the use of 36 drying beds of the designed size, which require a total land area of 4500 m^2 (0.45 hectares). The sludge drying beds will be constructed with roads for access for trucks and other large vehicles involved in their cleaning and/or maintenance.

Chapter 8 - Conclusion

Chemically enhanced primary treatment is a viable and cost-effective method of treating wastewater in developing countries.

The goal of the project was to design a facility which would achieve the effluent BOD_5 standard of 60 mg/L in the current land area the CEAGESP facility inhabits. Each design fulfills these goals. A brief analysis of each design alternative will proceed.

Alternative 1 Analysis

This aerated lagoon system designed by SABESP has an expected removal efficiency of BOD_5 95%, and will necessitate only two workers to operate the facility. The facility is characterized by high O&M costs due to the surface aerators (electricity and repair), and has a high capital cost, mostly incurred because of the purchase of 20 surface aerators in the first stage and 10 more surface aerators in the second stage of expansion.

Alternative 2 Analysis

Upon a 50% reduction of BOD_5 in the CEPT basins, the BOD_5 is further reduced by at least 50% in the anaerobic lagoon and again by at least 50% in the facultative lagoon, with an overall system removal efficiency of 88%. The lagoons were designed according to standards and if properly operated will achieve a high removal of BOD_5, potentially even higher than 88% as the BOD_5 removal in each lagoon is a very conservative estimate. The system's main cost will come from composting the sludge, whereas chemical addition is a minor O&M cost. The overall system efficiency would produce (empirically) an effluent BOD_5 of approximately 30 mg/L.

Alternative 3 Analysis

Alternative 3 is designed to have a BOD_5 removal equal to that of Alternative 2. And, since it doses only 30 mg/L in a facility with relatively no mechanical equipment, the O&M costs are minimal.

Alternative 3 is the optimum design due to its high BOD_5 removal and low cost. It is superior to Alternative 2 because of its lower O&M cost (with an equivalent capital cost), and is superior to Alternative 1 as this alternative is speculative whether or not it will achieve its predicted BOD_5 removal of 95%.

Appendix A-1

Equations for calculating ion distribution in the diffuse double layer

The following equations are used for calculating the ion distribution in the diffuse part of the electric double layer (Sonntag *et al.,* 1972):

The Poisson equation:

$$\mathrm{div}[\varepsilon(x)\mathrm{grad}\psi(x)] = -4\pi\rho(x)$$

The Boltzmann equation:

$$n_i = n_i(\infty)\exp\left(\frac{-W_i(x)}{kT}\right)$$

The equation for the space charge density:

$$\rho(x) = \sum_i z_i n_i(x)$$

where:

ε = The dielectric constant in the bulk of the solution

ψ = The potential

π = The space charge density

n_i = The number of ions of species *i* per cm^3

x = The distance from the interface

$W_i(x)$ = The work required to transfer an ion *i* from the bulk of the solution (x= infinity) to a distance x from the interface

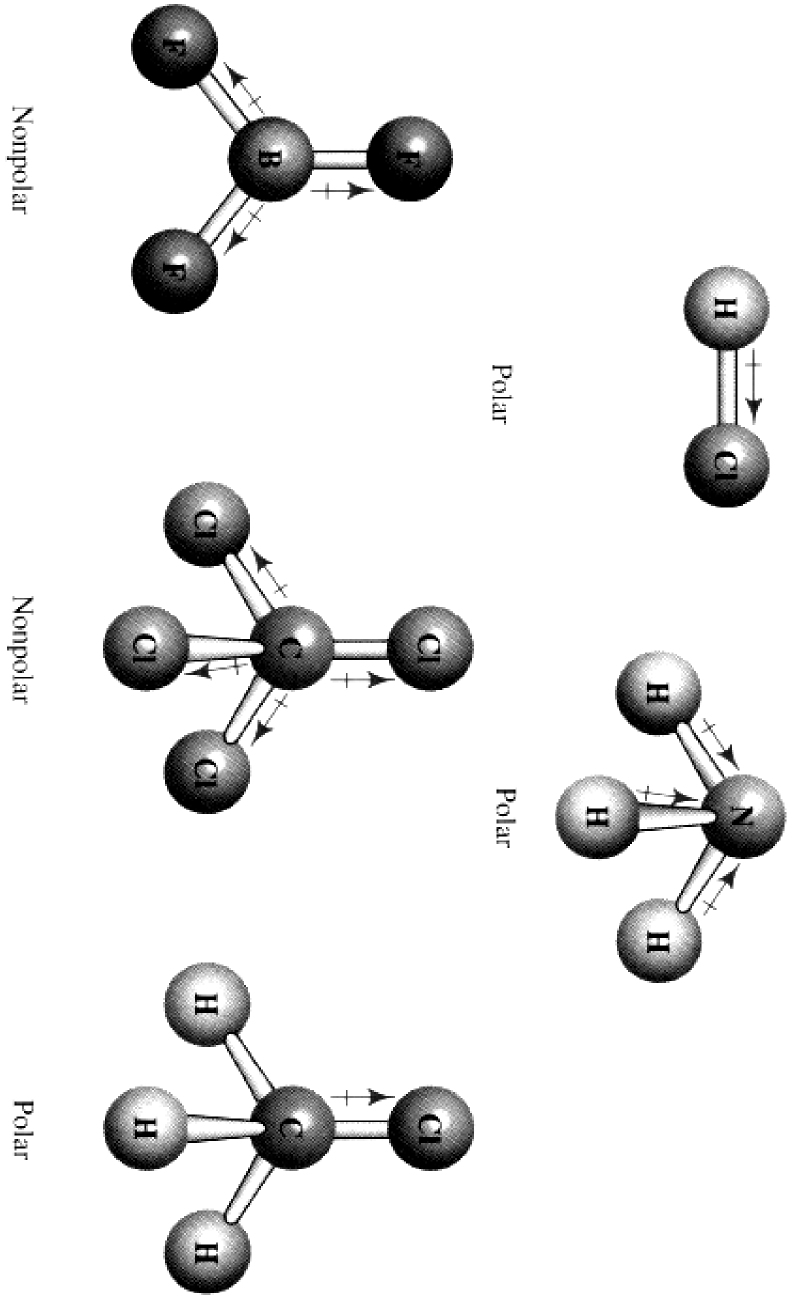
e = The elementary charge

k = The Boltzmann constant (=1.38 x 10-16 erg/degree)

T = Temperature

Appendix A-2

Polarity



The following information is form Brown *et al.*, 1994:

A molecule is said to be polar if its centers of negative and positive charge do not coincide. Because one end of a polar molecule has a slight negative charge and the other a slight positive charge, polar molecules are also called dipoles. If there are no charges on the opposite ends of a molecule, or if the charges have the same sign, the molecule is not a dipole and is therefore nonpolar.

The degree of polarity of a molecule is measured by its dipole moment. The dipole moment, μ, is defined as the product of the charge at either end of the dipole, Q, times the distance, r, between the charges: $\mu = Qr$.

The following are examples of molecules with polar bonds. Some of these molecules are nonpolar because their bond dipoles cancel one another.

Appendix A-3

Chemicals Used in Wastewater

			Density, lb/ft^3	
Chemical	Formula	Molecular Wt	Dry	Liquid
Alum	$Al_2(SO_4)_3 \cdot 18H_2O$	666.7	60-75	78-80 (49%)
	$Al_2(SO_4)_3 \cdot 14H_2O$	594.3	60-75	83-85 (49%)
Ferric Chloride	$FeCl_3$	162.1		84-93
Ferric Sulfate	$Fe_2(SO_4)_3$	400		
	$Fe_2(SO)_4 \cdot 3H_2O$	454		70-72
Ferrous Sulfate	$FeSO_4 \cdot 7H_2O$	278	62-66	
Lime	$Ca(OH)_2$	56 as CaO	35-50	

Source: Metcalf & Eddy (1991)

Governing Reactions

Ferric Sulfate:

$$Fe2(SO_4)_3 + 3Ca(HCO_3)_2 \rightarrow \underline{2Fe(OH)_3}\downarrow + 3CaSO_4 + 6CO_2$$

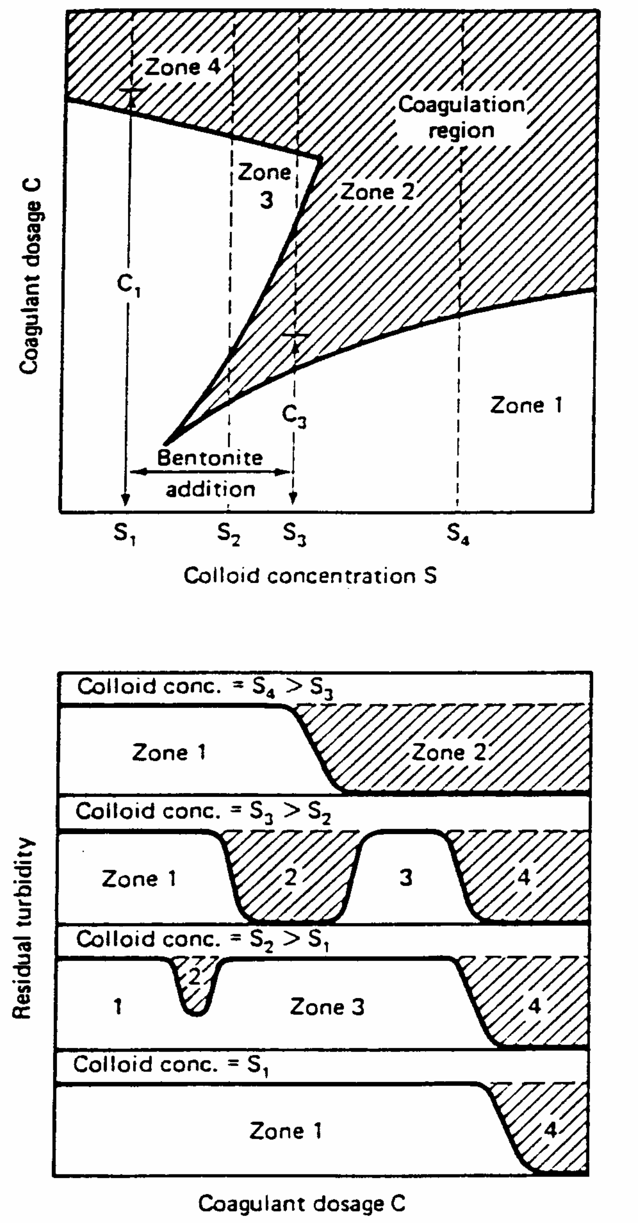
Ferric Chloride:

$$2FeCl_3 + 3Ca(HCO_3)_2 \rightarrow \underline{2Fe(OH)_3}\downarrow + 3CaSO_4 + 6CO_2$$

Source: Reynolds *et al.*, 1996.

Appendix A-4

Colloid Concentration and Coagulation Dosage Relationship



Source: Benefield *et al*, 1992.

Appendix A-5

van der Waals Equations

The three equations for van der Waals attraction forces (Hiemenz, 1986):

The DeBye equation:

$$\Phi_D = -\frac{\left(\alpha_2\mu_1^2 + \alpha_1\mu_2^2\right)}{4\pi\varepsilon_0}x^{-6}$$

The Keesom equation:

$$\Phi_K = -\frac{2}{3}\frac{\mu_1^2\mu_2^2}{k_B T\left(4\pi\varepsilon_0\right)^2}x^{-6}$$

The London equation:

$$\Phi_D = \frac{1}{2}h\left[\left(v_i + v_j\right) - 2v\right]$$

where:

z = Valence
e = Electron
μ = Dipole moment
v = Characteristic vibrational frequency of electrons (always negative)
h = Planck’s constant
c = Speed of light

Appendix A-6

Coalescence

The breakup of coalescence-stable layers was obtained form Sonntag *et al.*'s (1972) analysis of emulsions and foams. This must be kept in mind in the following discussion of coalescence. This paper will not present a direct correlation and/or comparison between this information and what would be expected in a sol, such as what is encountered in water and wastewater treatment.

As previously pointed out, the adsorption layers of colloids (particles) must be broken to enable coalescence to occur. It can occur in three ways, according to Sonntag *et al*. (1972) data. One way is when external conditions create the desorption of the stabilizers. The dispersion can be broken by the addition of surfactants which will preferentially adsorb at the interface; these do not form coalescence-stable layers. The second way is when the adsorption layer undergoes chemical changes, which leads to the formation of a new structure. The third means is when there is an addition of mechanical strength to the system which is greater in energy than the mechanical strength of the adsorption layer. This is a reason why gentle mixing is utilized for the flocculation process in water and wastewater treatment. Rapid mixing can apply a strong shearing stress which can exceed the strength of the adsorption layer, which will cause the dispersion to break up.

Appendix B

Aerobic Sludge Digester Aeration Capacity: (based on Table 12-24 and Example 12-7 from Metcalf & Eddy, 1991)

Oxygen required: 1.8(lbO_2/lbsolid) (assumption)

Air: Thod/(γ_{air}*%O_2) see appendix B

γ_{air}= 0.0752 lb/ft^3 %O_2 = 23.2%

SOTE for ceramic disc= 32% Safety Factor (SF) = 2

Loadings:

Average

Solids= 6649.2 lbs TSS/d 3022 kg TSS/d

Avg: 4787.42 lbs O_2/d multiply here by 0.4 since a 40% reduction in VS is the goal

air: 274407 ft^3/day (w/o SF of 2 and SOTE of 32%)

required air: 1715046 ft^3/day

$OTR_f = SOTR[(\beta C_s - C_w)/C_{s20}]\theta^{(T-20)}(\alpha)$ (Eqn. 6-64)

T=	20	30	Celcius	
SOTR =	3	3	lb/bhp-hr	(asssumed spec for aerator)
β=	0.95	0.95		(p.286)
α=	0.8	0.8		(p.286)
C_s=	9.08	7.54	mg/l	(Appendix E)
C_w=	2	2	mg/l	(Assumed)
C_{s20}=	9.08	9.08	mg/l	(Appendix E)
θ=	1.024	1.024		(p.286)

OTR_f= 1.751366 lbO_2/bhp-hr at 20 Celcius

OTR_f= 1.729924 lbO_2/bhp-hr at 30 Celcius

Necessary break horsepower:

bhp = (O_2 demand)/(OTR_f*24)

113.8973 bhp at 20 C and Avg conditions

115.309 bhp at 30 C and Avg conditions

This breakhorsepower demand will result in high O&M costs due to the high use of electricity, thus it is not a feasible option for the CEAGESP facility in Tatui. Since this was not a chosen option, no detailed description of calculations will be shown. It should be noted that little information exits as to the applicability of aerobic digestion to chemical sludge. Thus, this is more of a conceptual design, a design which is very vulnerable to changes in SOTR and C_w.

Appendix C - Jar Test Data

This appendix is a collection of the data obtained from the Jar Tests and other lab work done in Tatui, Brazil in January, and its subsequent analysis. The analysis involved directly in the design is presented throughout the thesis, but, the analysis of other data is not. That is, this appendix contains plots and data which were analyzed, but an explanation of the analysis is not presented. Other plots included chemical coagulant comparisons and the data from the 24-hr lagoon sampling test.

Terms

This section will explain the terms used in the data in the subsequent pages.

ID#: This is to follow data from page to page.

Date: The date the test/experiment was done.

Time: The time at which the test/experiment was done. NA will refer to all data that is Not Available.

Sampling Location: Where the sample came from. “Splitter box” is where the flow is split to the three pipes into the anaerobic lagoon, and the overflow into the river. “B1” is the head of the anaerobic lagoon. “G1” is the effluent of the anaerobic lagoon. “J1” is the effluent of the facultative lagoon.

Sampling Time: The time of sampling.

Sample (Jar#): When this is a number, it is the jar number in the jar test (labeled 1 through 4, form the left-most jar, to the right-most jar). “Raw” means a raw sample, one which did not undergo any jar test procedure. B1, J1, and G1 refer to the same as in sampling location. For the ones labeled 1min, 2min…5min, this means the exact minute of sampling during the settling tests.

Coagulant: The specific coagulant used. Zero means the “zero jar” in which no coagulant was added although the sample was part of a jar test run. “Sl” indicates sludge (such as “NHEEL + Sl.”

Chemical Dosage: The amount of chemical dosed to the jar.

Polymer: The type of polymer dosed. None means no polymer was injected. “S-non” is a non-ionic polymer provided by SABESP, as “S-cat” is a cationic and “S-an” is an anionic polymer, both provided by SABESP.

Polymer Dosage: The amount of polymer dosed to each jar.

COD: The measured COD in the sample.

COD %Removal: The amount of COD removal achieved by the jar test, as compared to a raw sample(s).

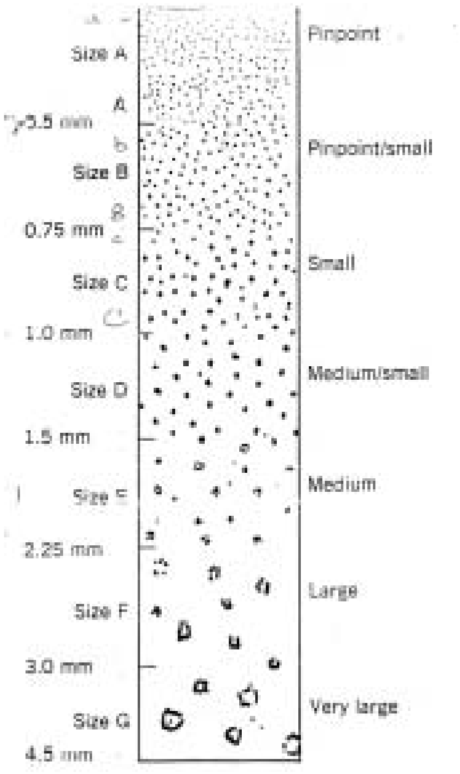
TSS: The measure TSS of the sample.

TSS %Removal: The amount of TSS removal achieved by the jar test, as compared to a raw sample(s).

SO_4: The detected amount of $S0_4$-sulfates in the sample.

PO_4: The detected amount of PO_4-phosphates in the sample.

Floc Size: A visual observation of floc as compared to the following figure:



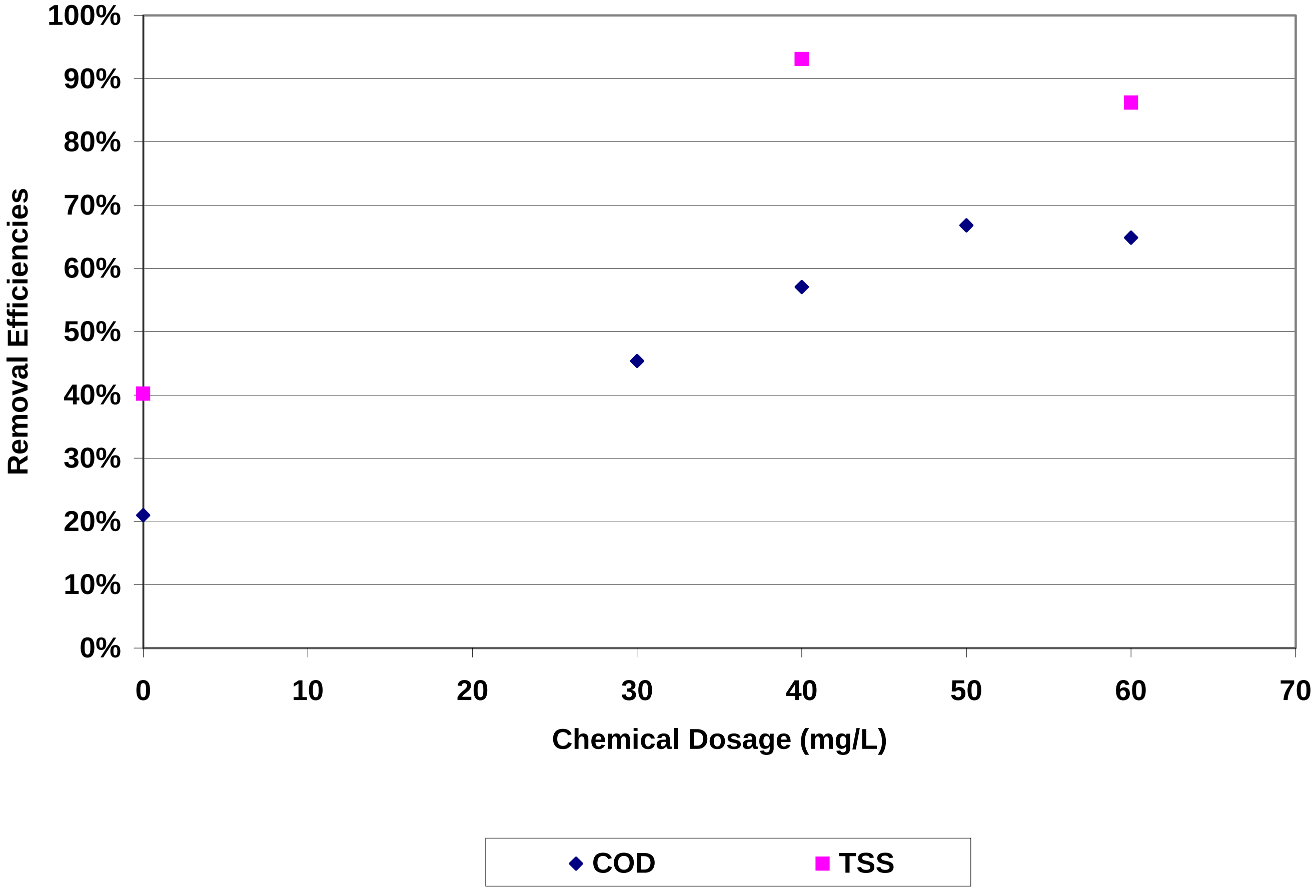
Visual Observation: That which was observed during the jar test or in a sample specimen.

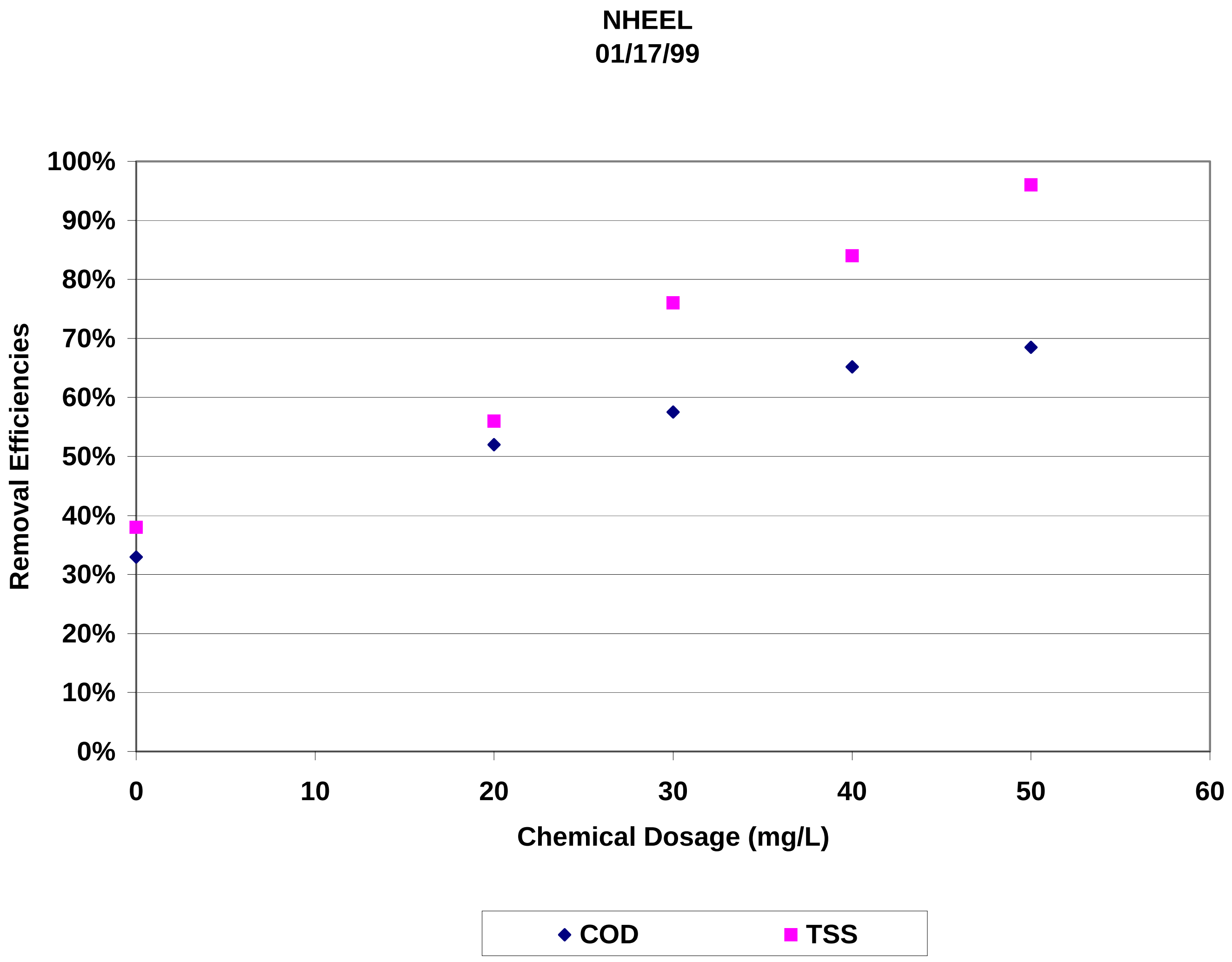
Sample Volume: The volume of wastewater in each jar.

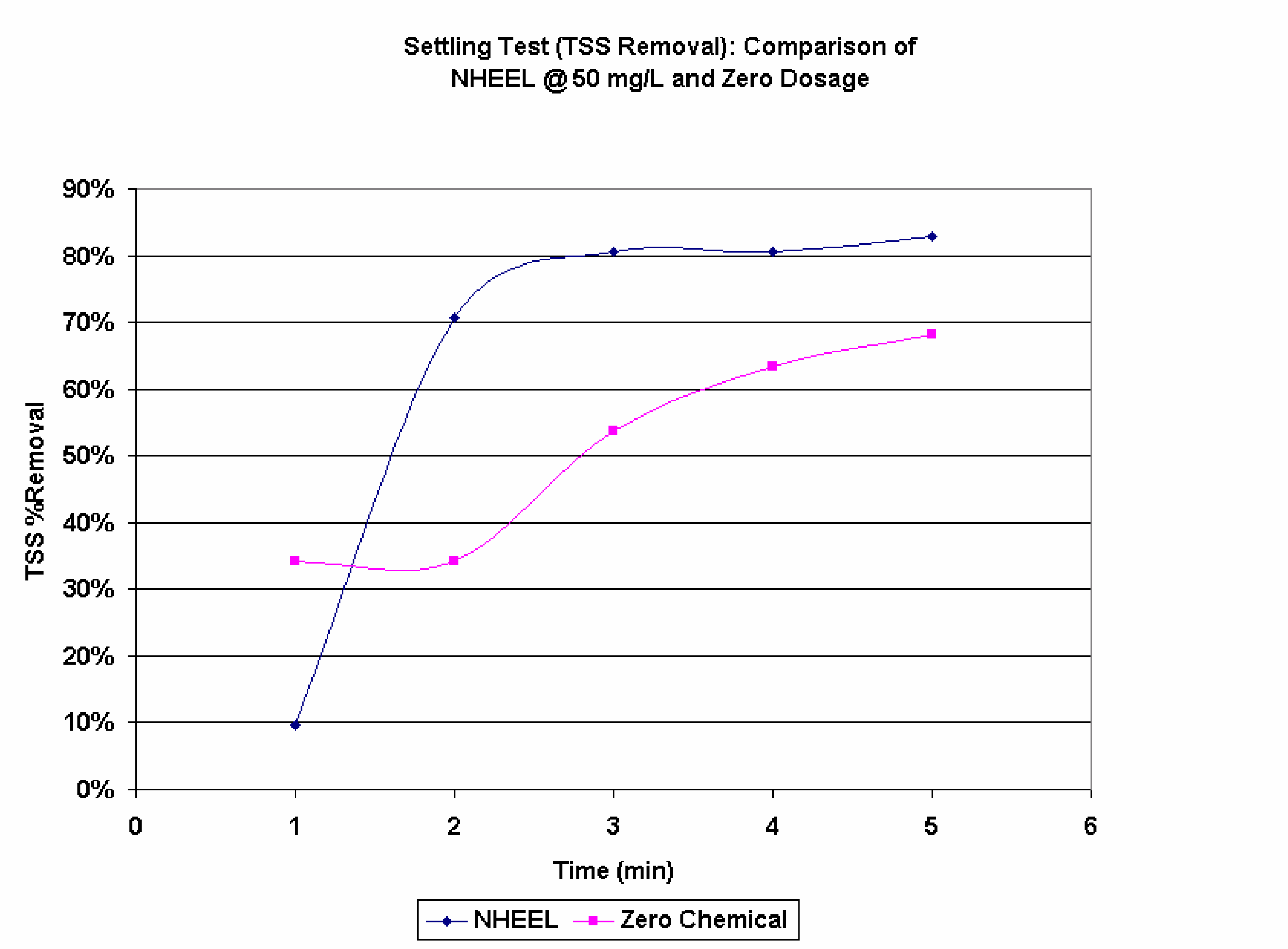
Purpose => Results: The purpose of the jar tests, and the observed results.

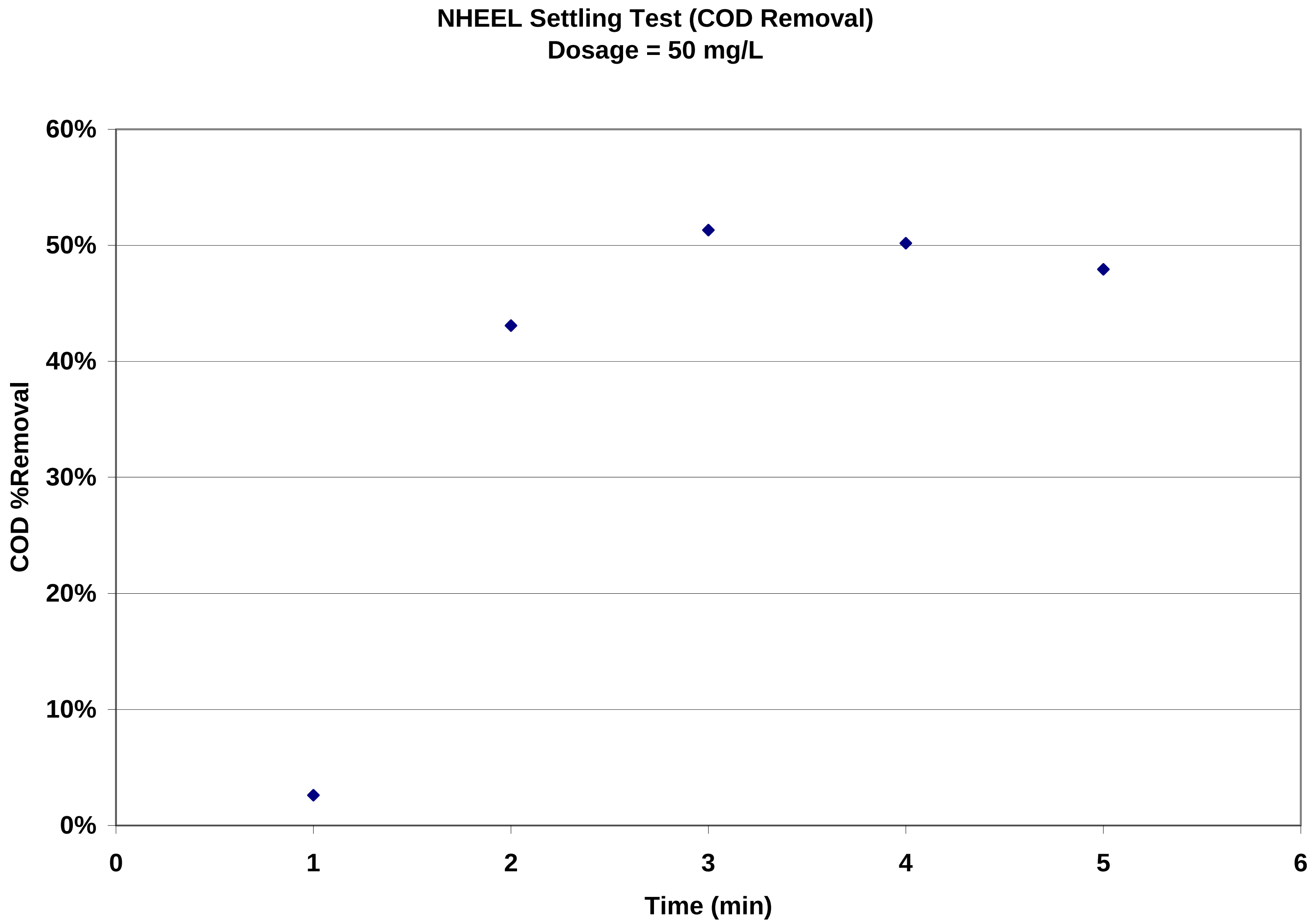
The plots follow and data follow.

NHEEL
01/16/99

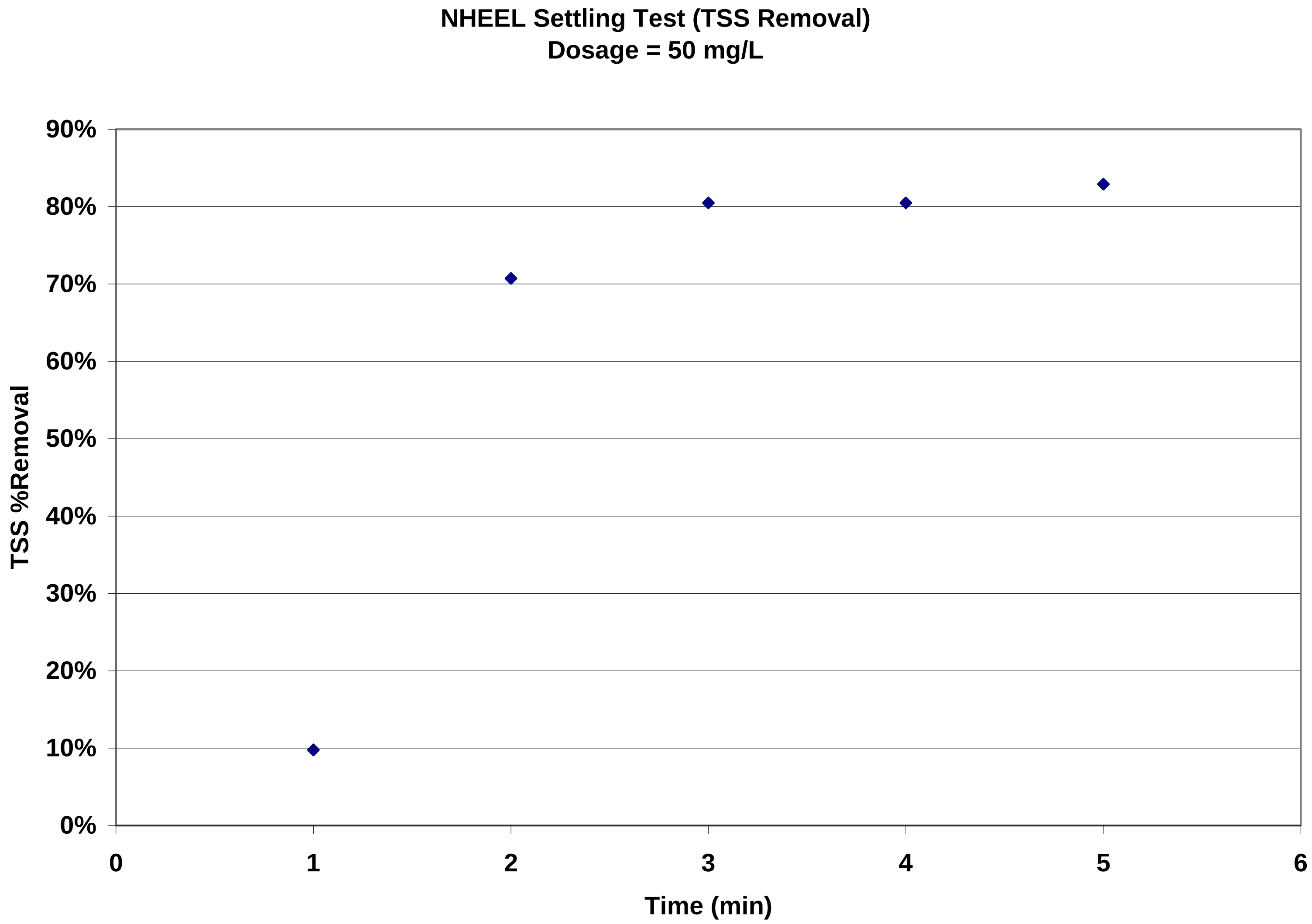



NHEEL
01/17/99
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Removal Efficiencies
0
10
20
30
40
50
60
Chemical Dosage (mg/L)
◆ COD
■ TSS

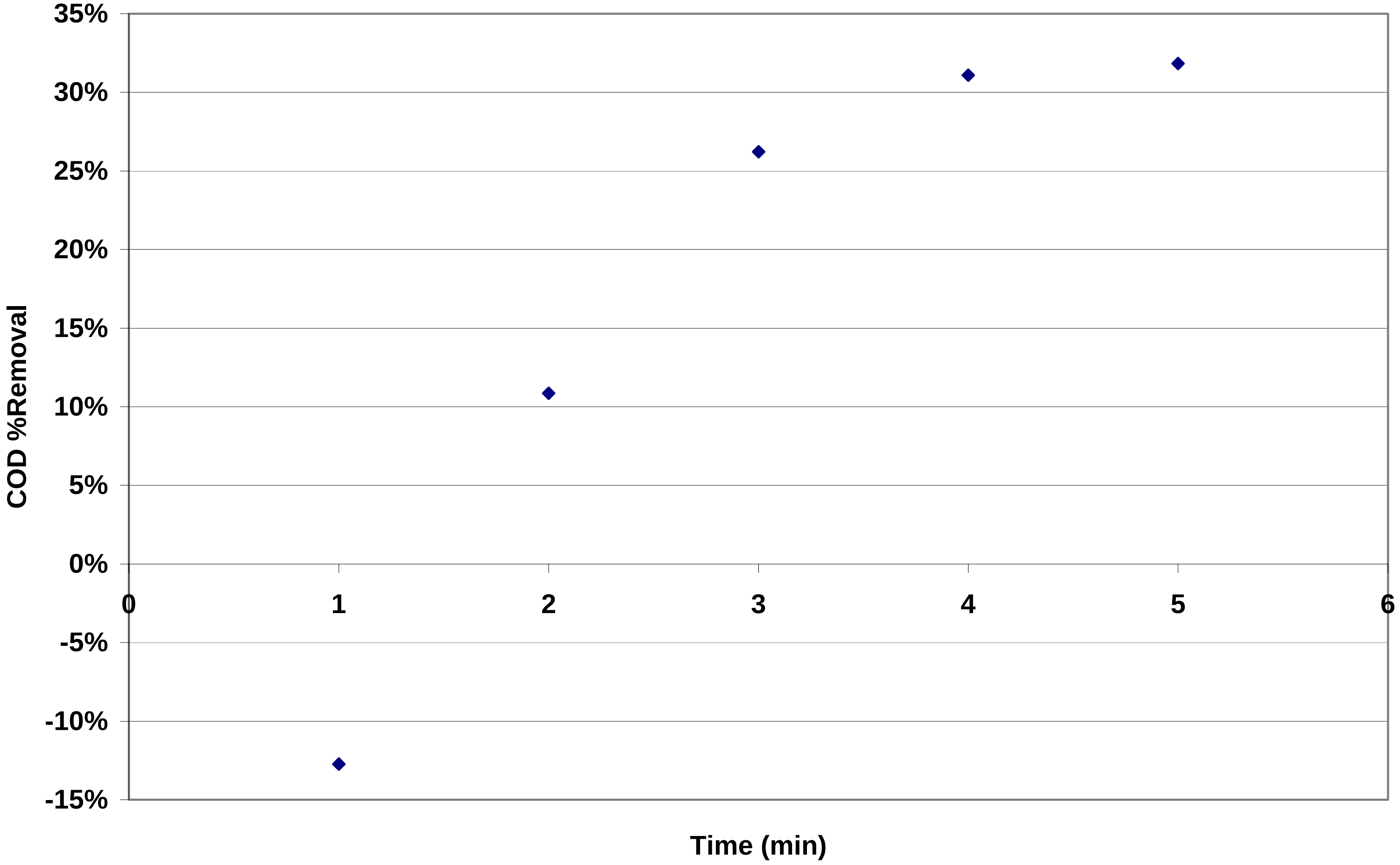

Settling Test (TSS Removal): Comparison of NHEEL @ 50 mg/L and Zero Dosage
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
TSS %Removal
0
1
2
3
4
5
6
Time (min)
NHEEL
Zero Chemical



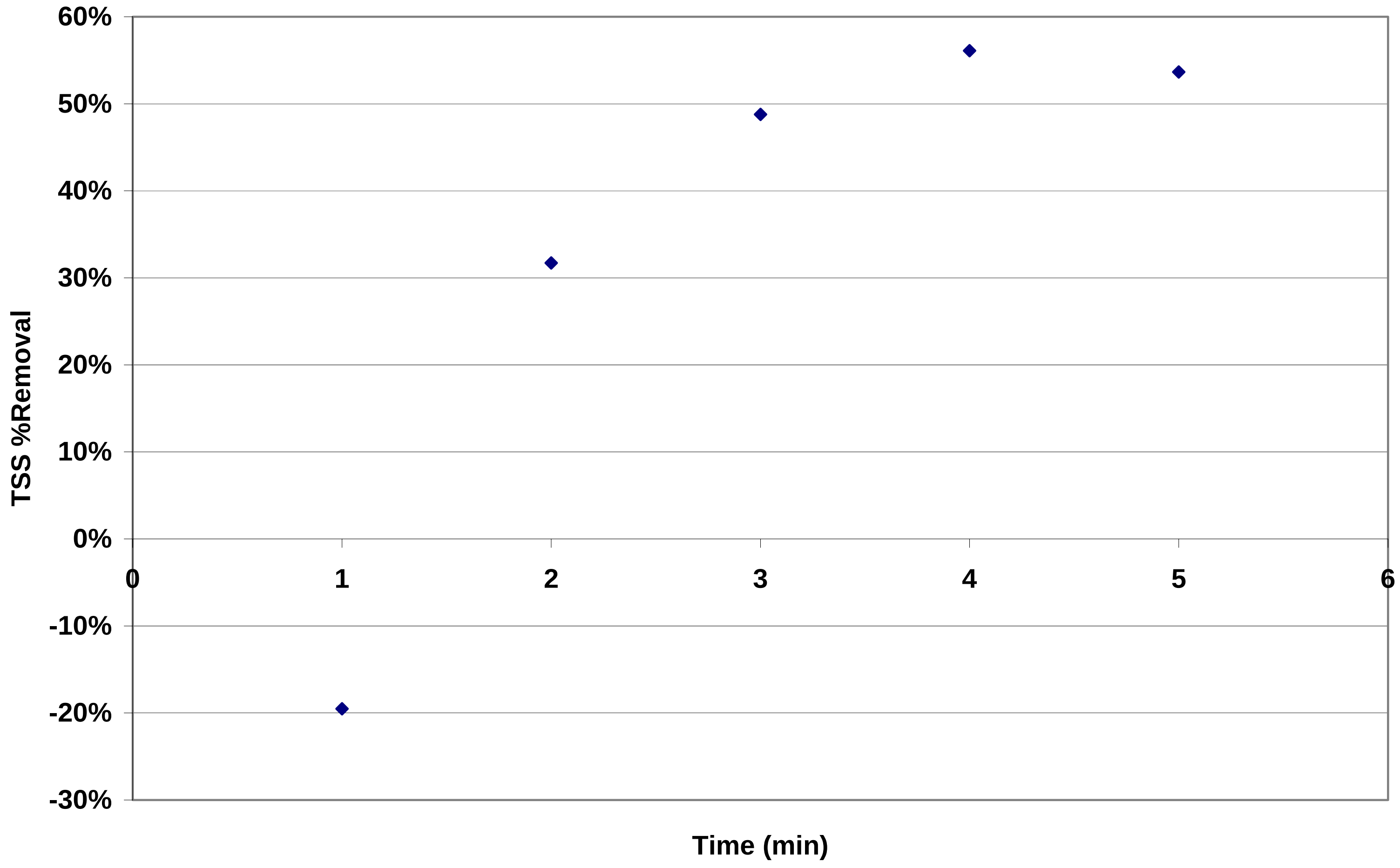
NHEEL Settling Test (COD Removal)
Dosage = 50 mg/L


NHEEL Settling Test (TSS Removal)
Dosage = 50 mg/L
TSS %Removal
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
1
2
3
4
5
6
Time (min)

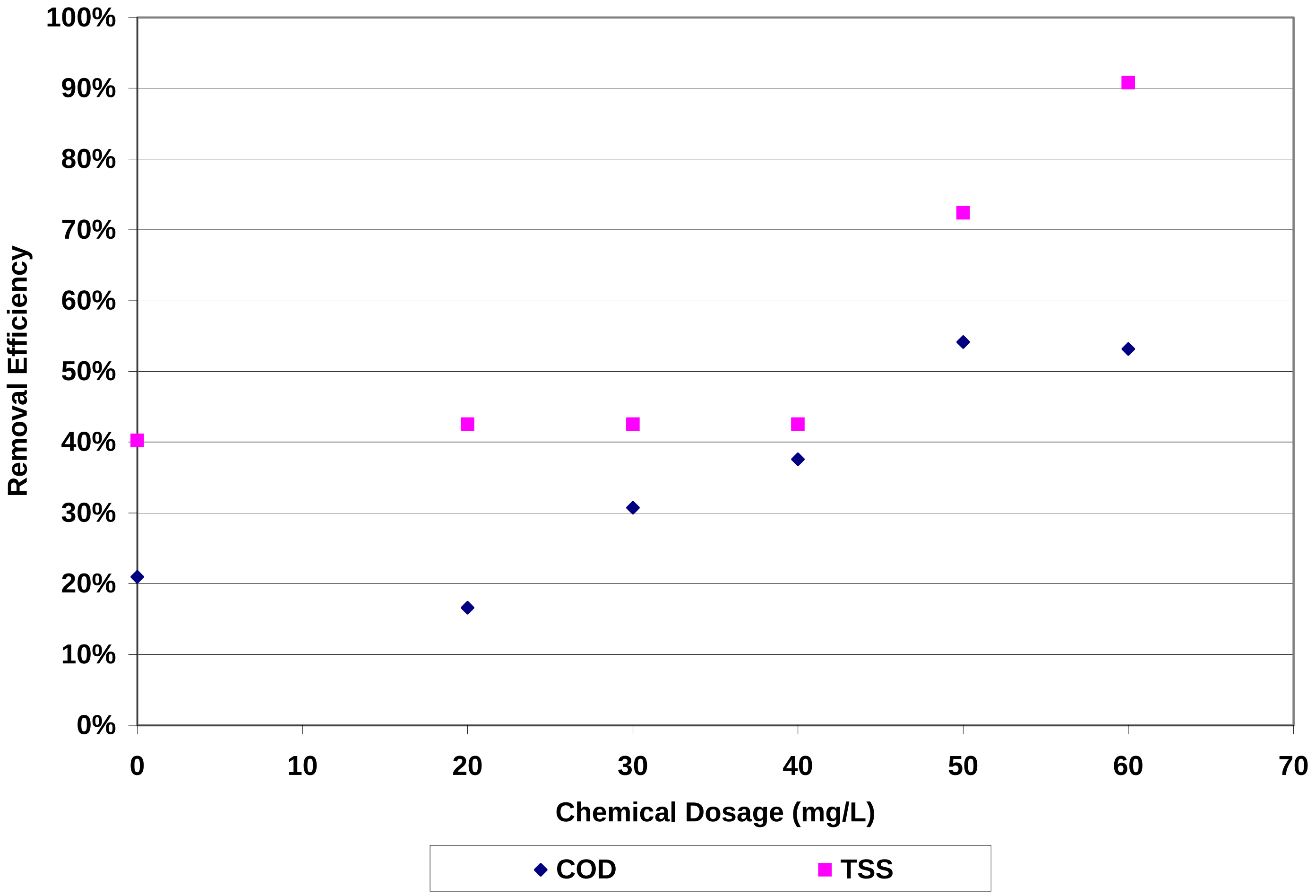
Sanechlor Settling Test (COD Removal)
Dosage = 50 mg/L



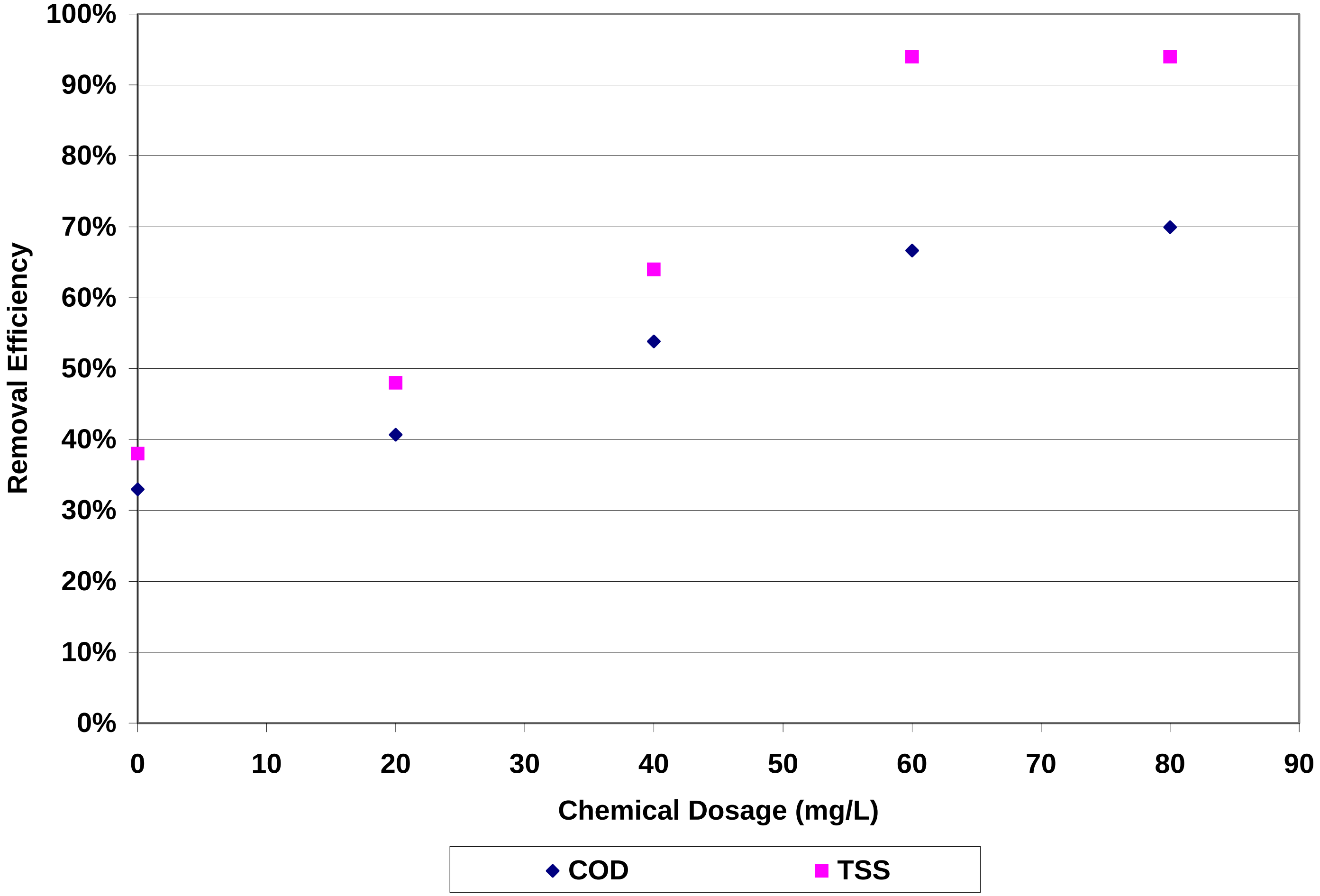
Sanechlor Settling Test (TSS Removal)
Dosage = 50 mg/L

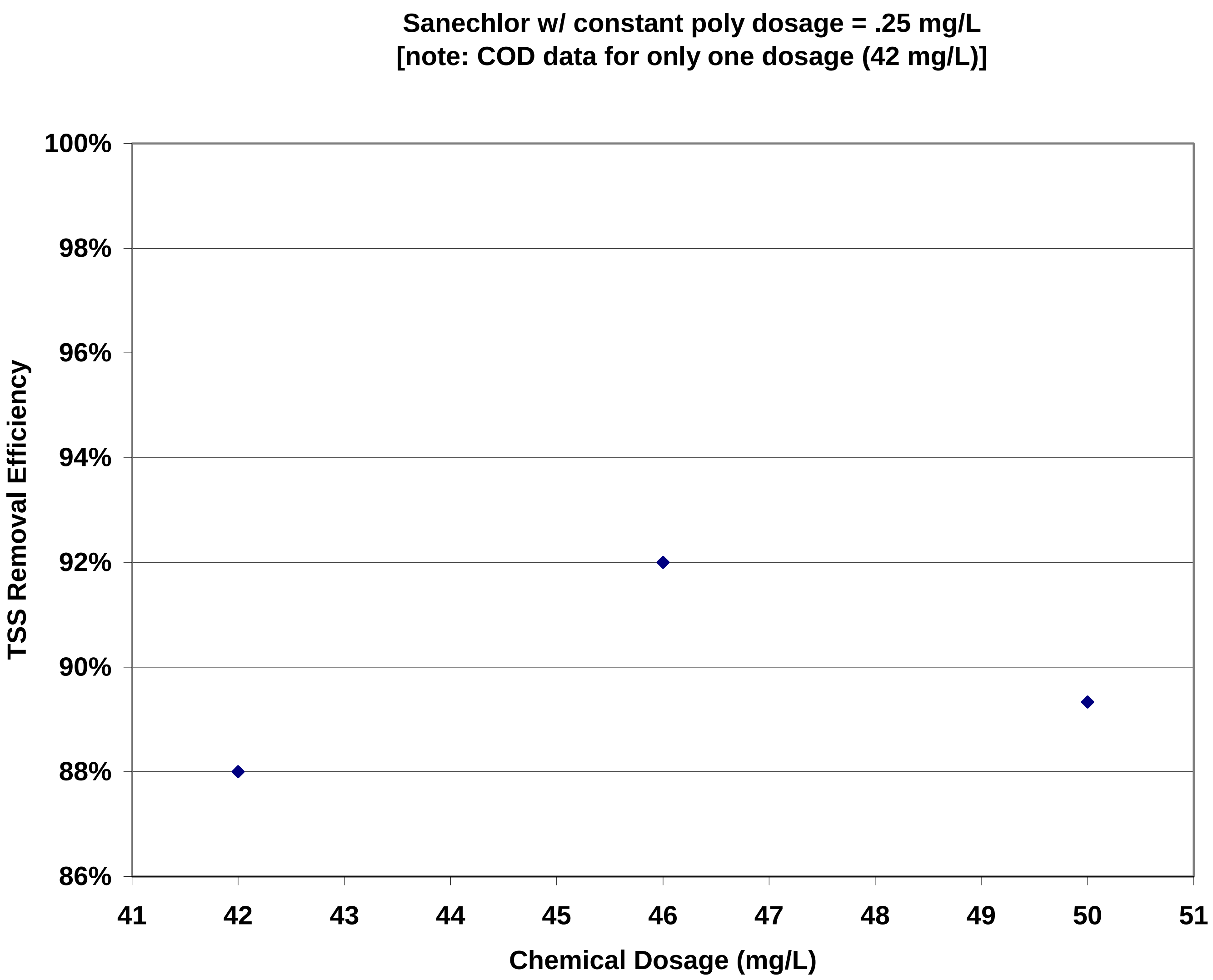


Sanechlor
01/16/99

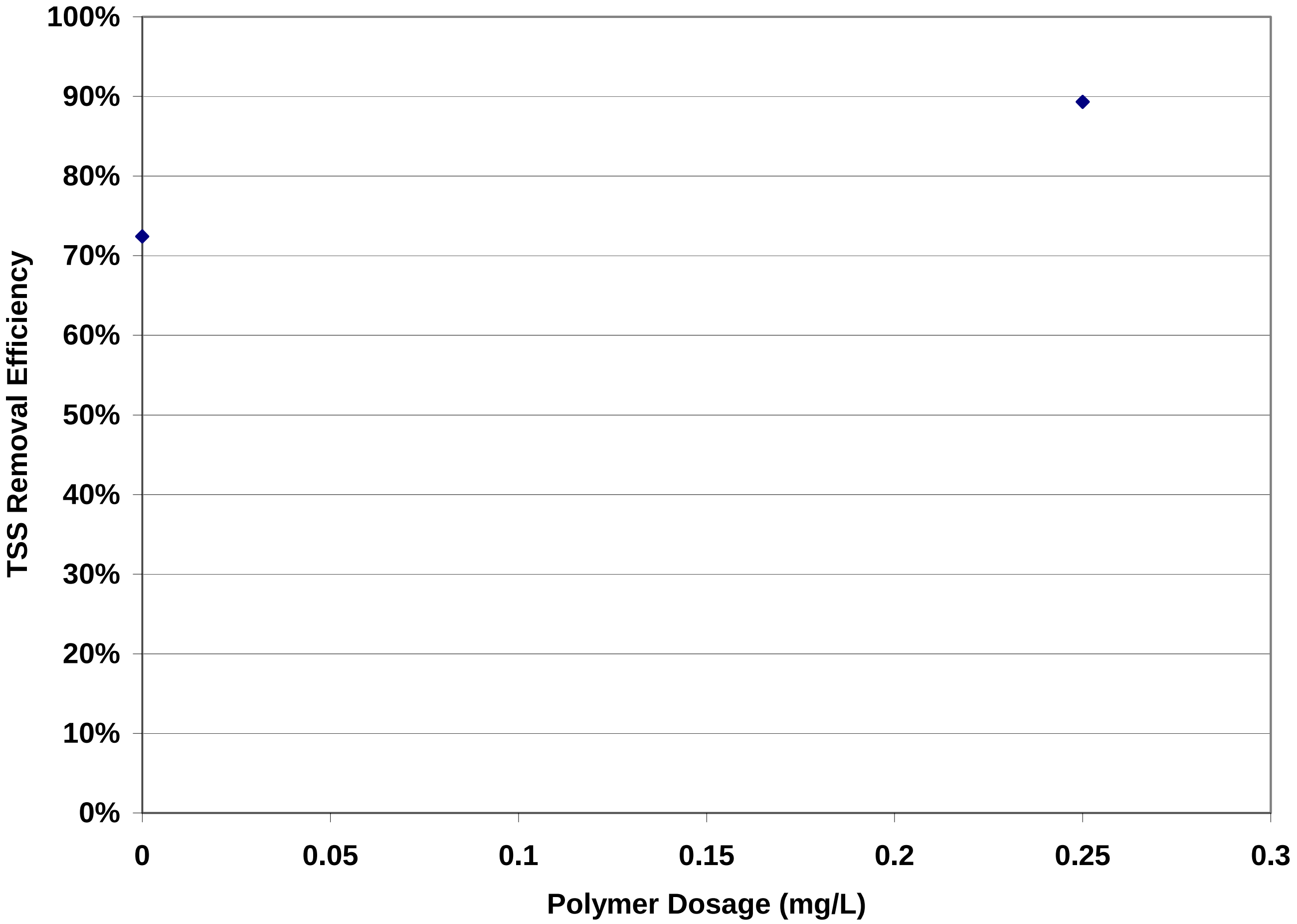


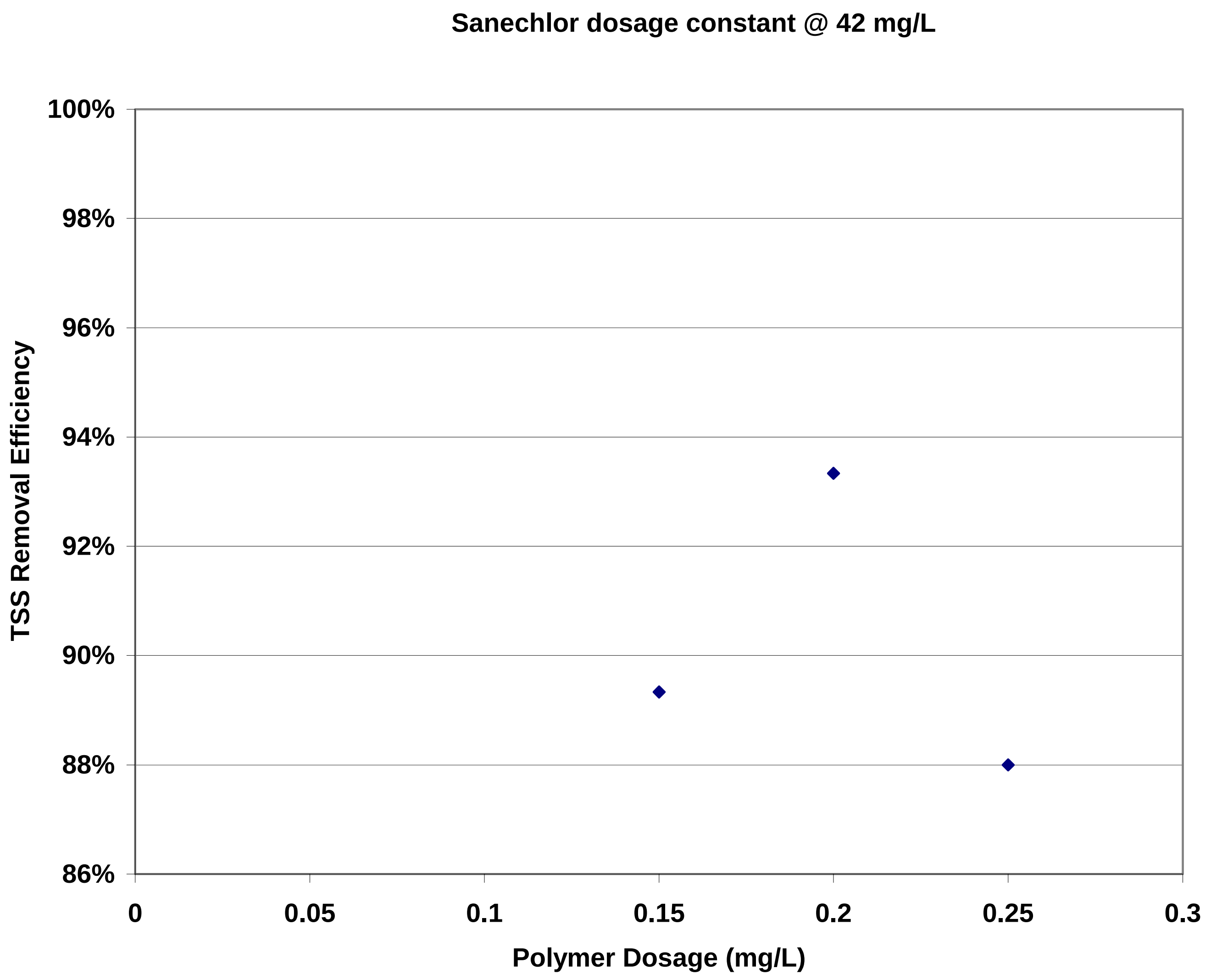
Sanechlor
01/17/99

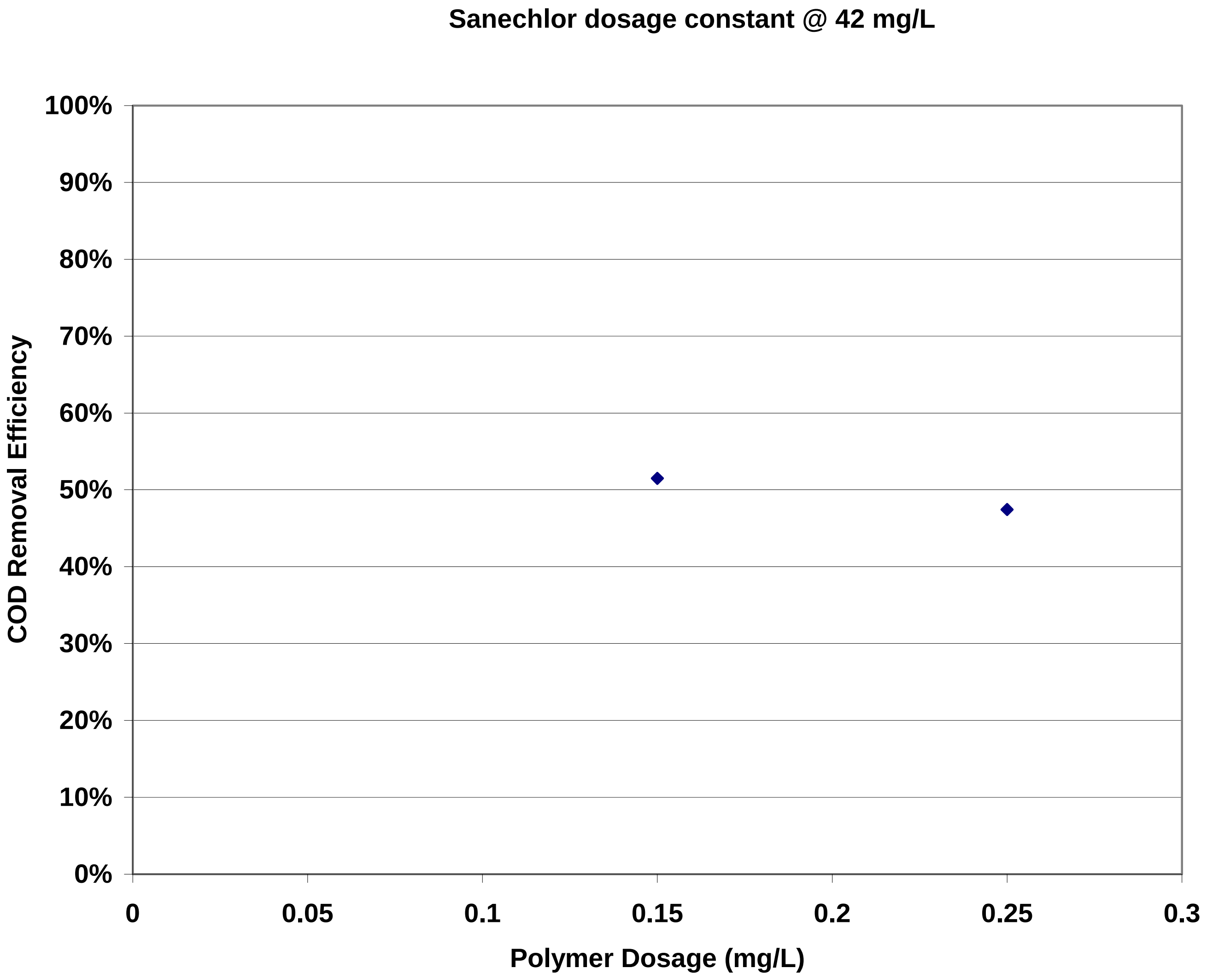


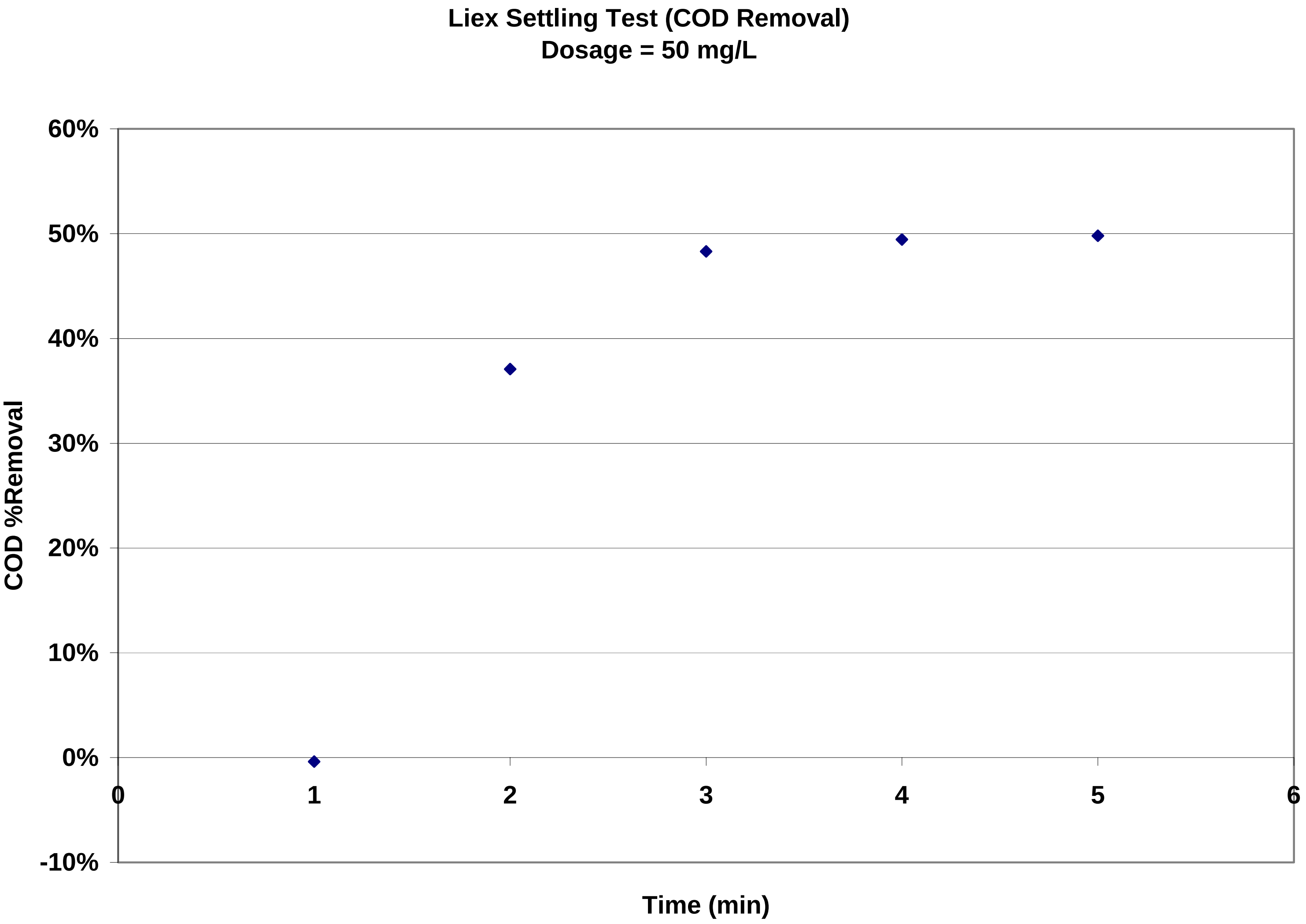

Sanechlor w/ constant poly dosage = .25 mg/L
[note: COD data for only one dosage (42 mg/L)]
TSS Removal Efficiency
100%
98%
96%
94%
92%
90%
88%
86%
41
42
43
44
45
46
47
48
49
50
51
Chemical Dosage (mg/L)

Sanechlor (50 mg/L) with and without polymer
[COD unavailable]

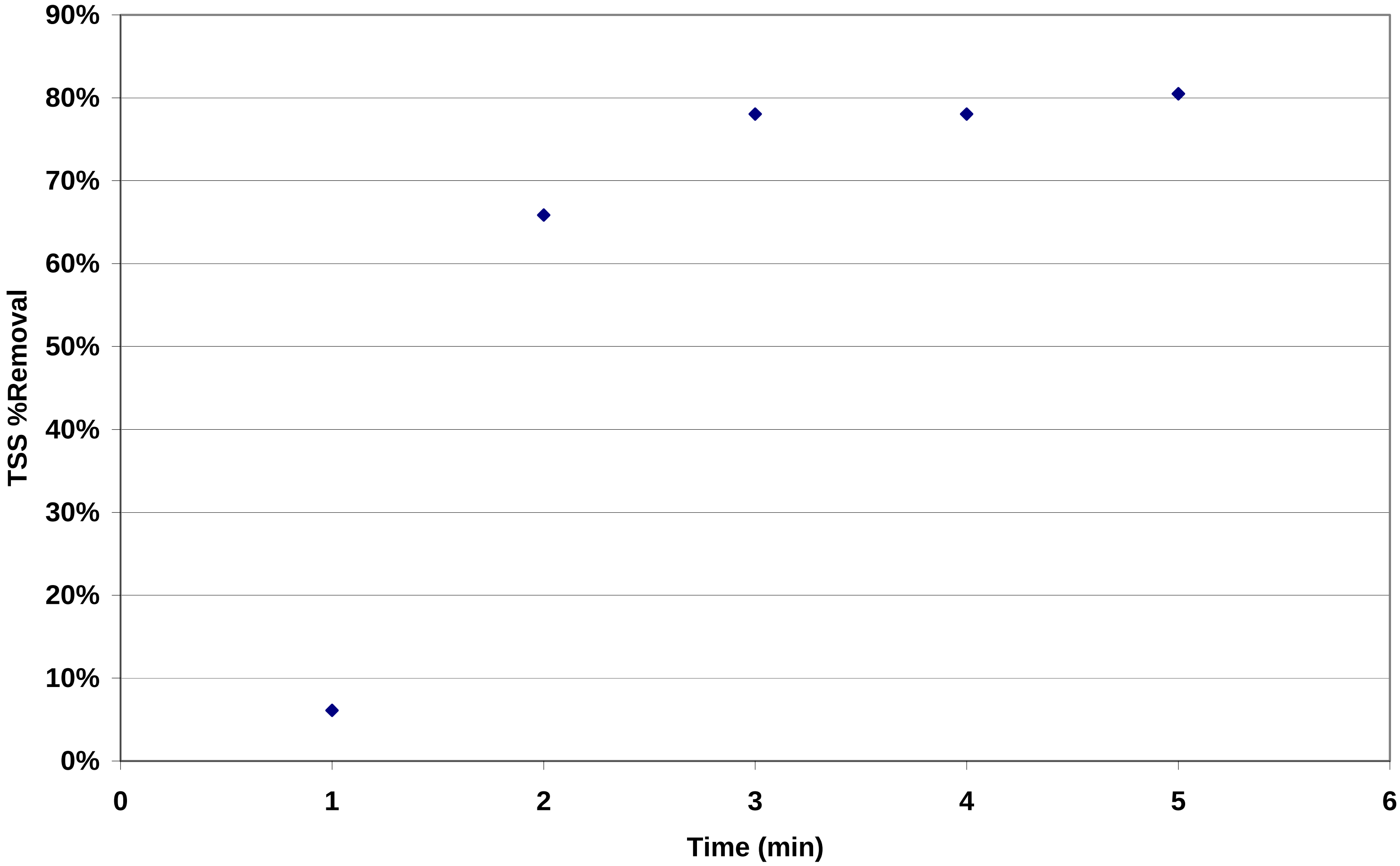


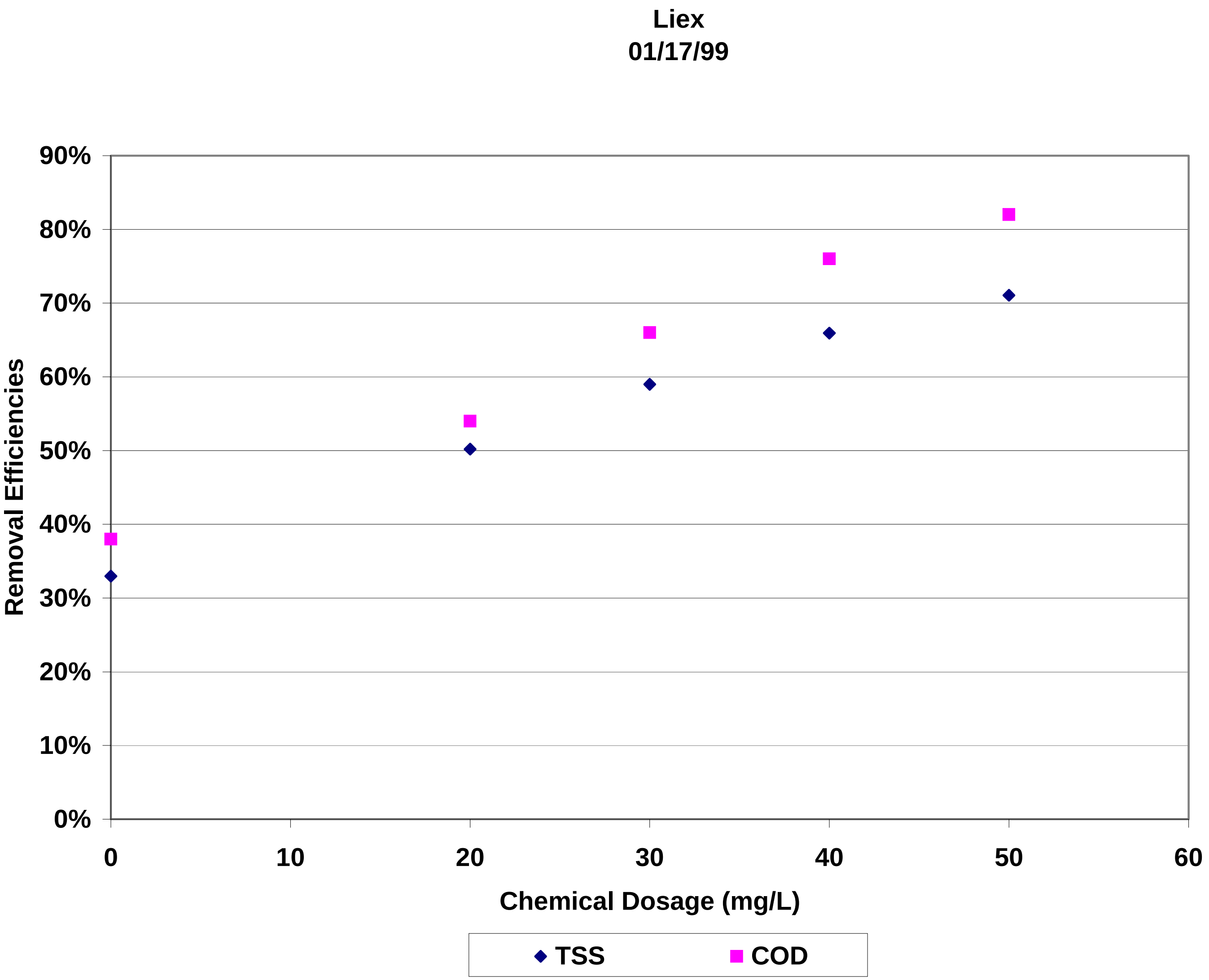

Sanechlor dosage constant @ 42 mg/L
100%
98%
96%
94%
92%
90%
88%
86%
TSS Removal Efficiency
0
0.05
0.1
0.15
0.2
0.25
0.3
Polymer Dosage (mg/L)

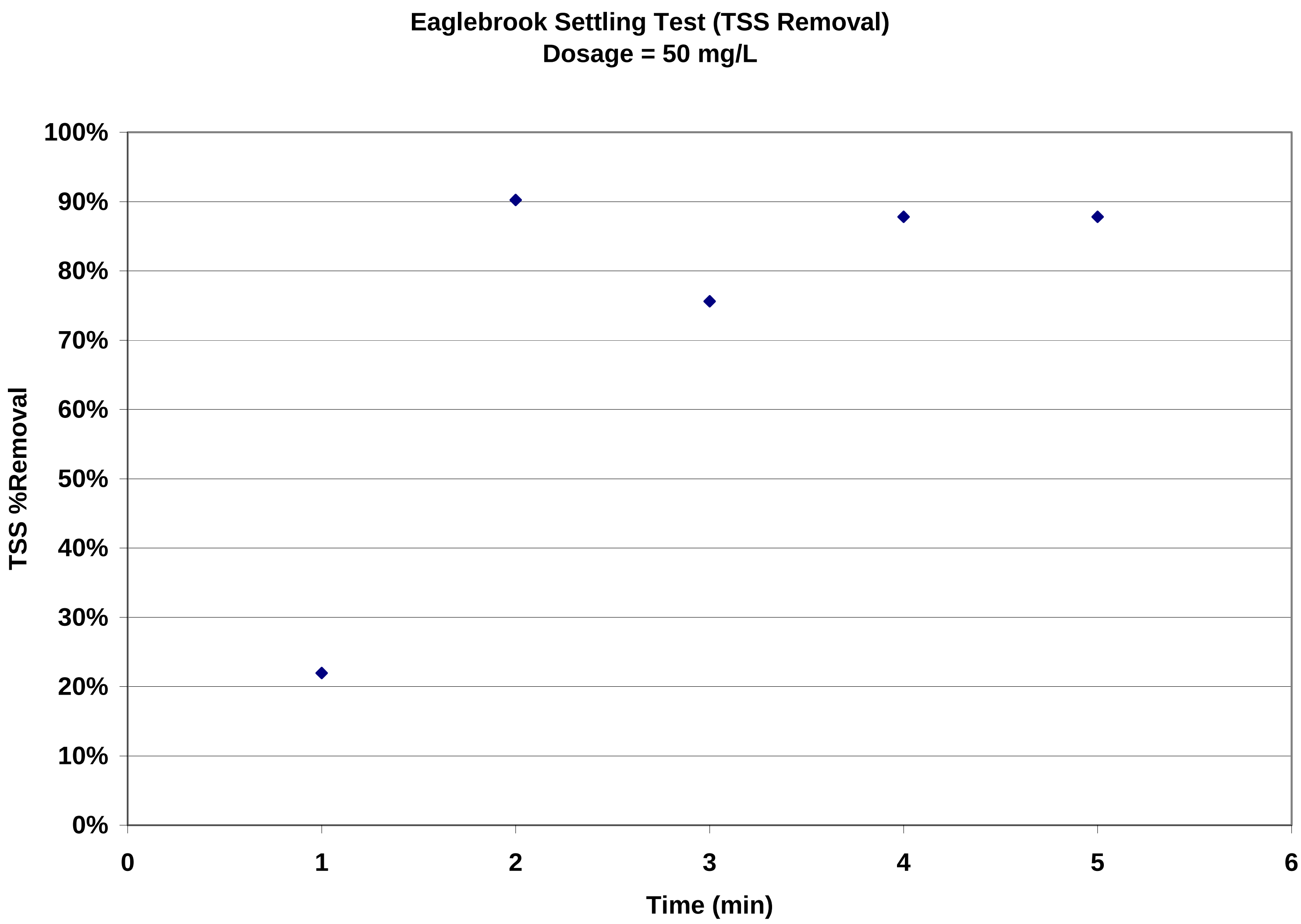

Sanechlor dosage constant @ 42 mg/L
COD Removal Efficiency
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
0.05
0.1
0.15
0.2
0.25
0.3
Polymer Dosage (mg/L)

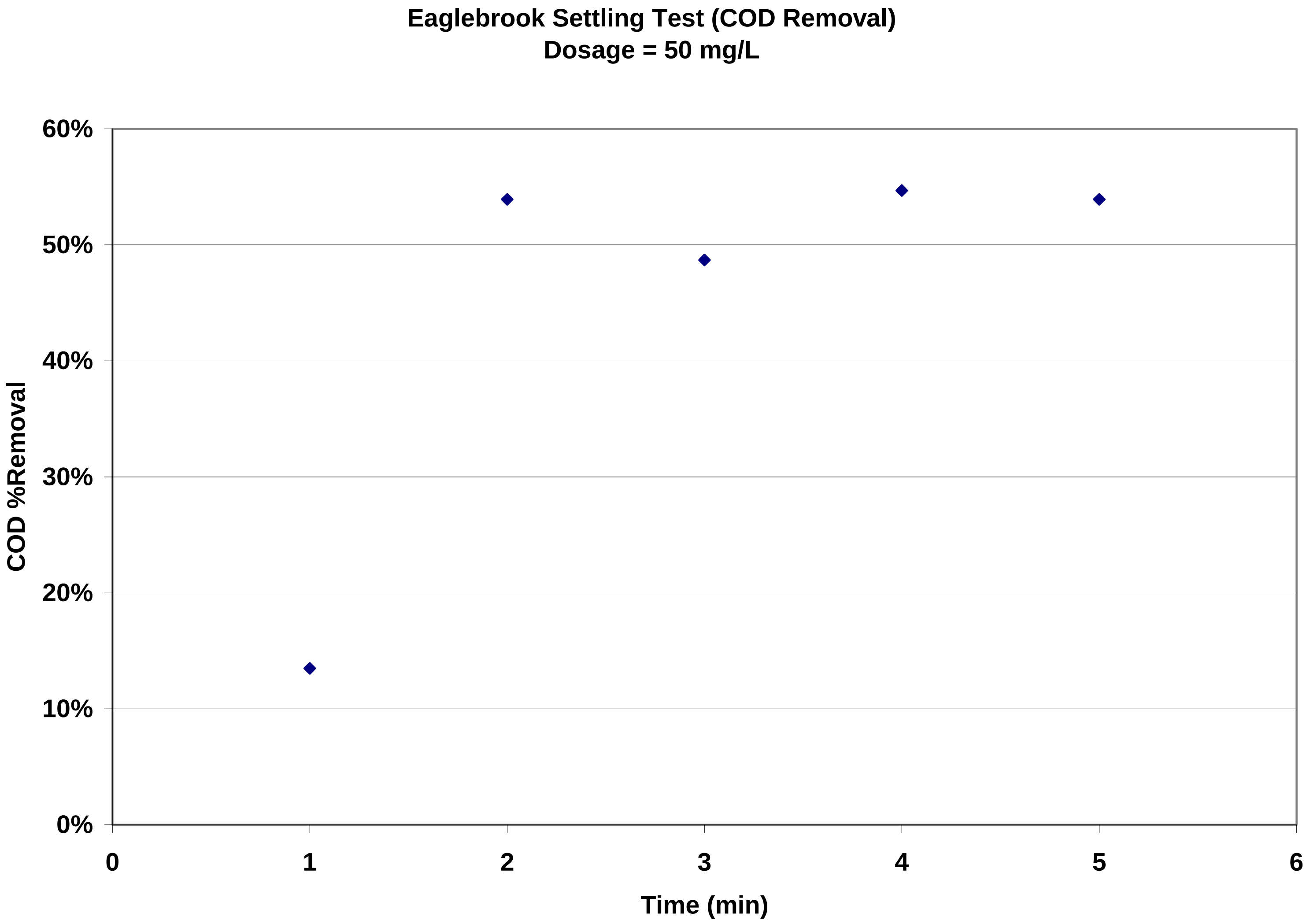

Liex Settling Test (COD Removal)
Dosage = 50 mg/L
COD %Removal
60%
50%
40%
30%
20%
10%
0%
-10%
0
1
2
3
4
5
6
Time (min)

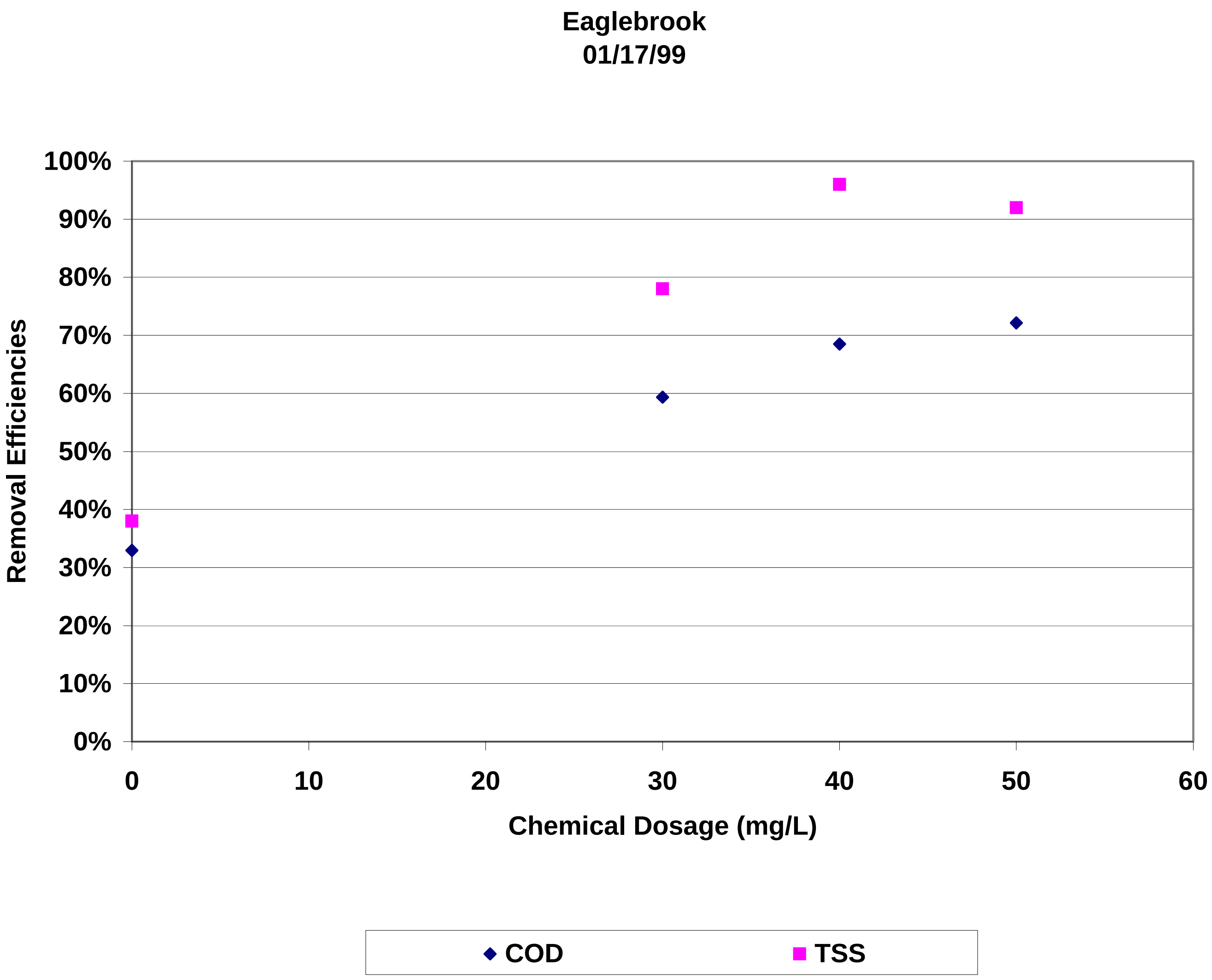
Liex Settling Test (TSS Removal)
Dosage = 50 mg/L

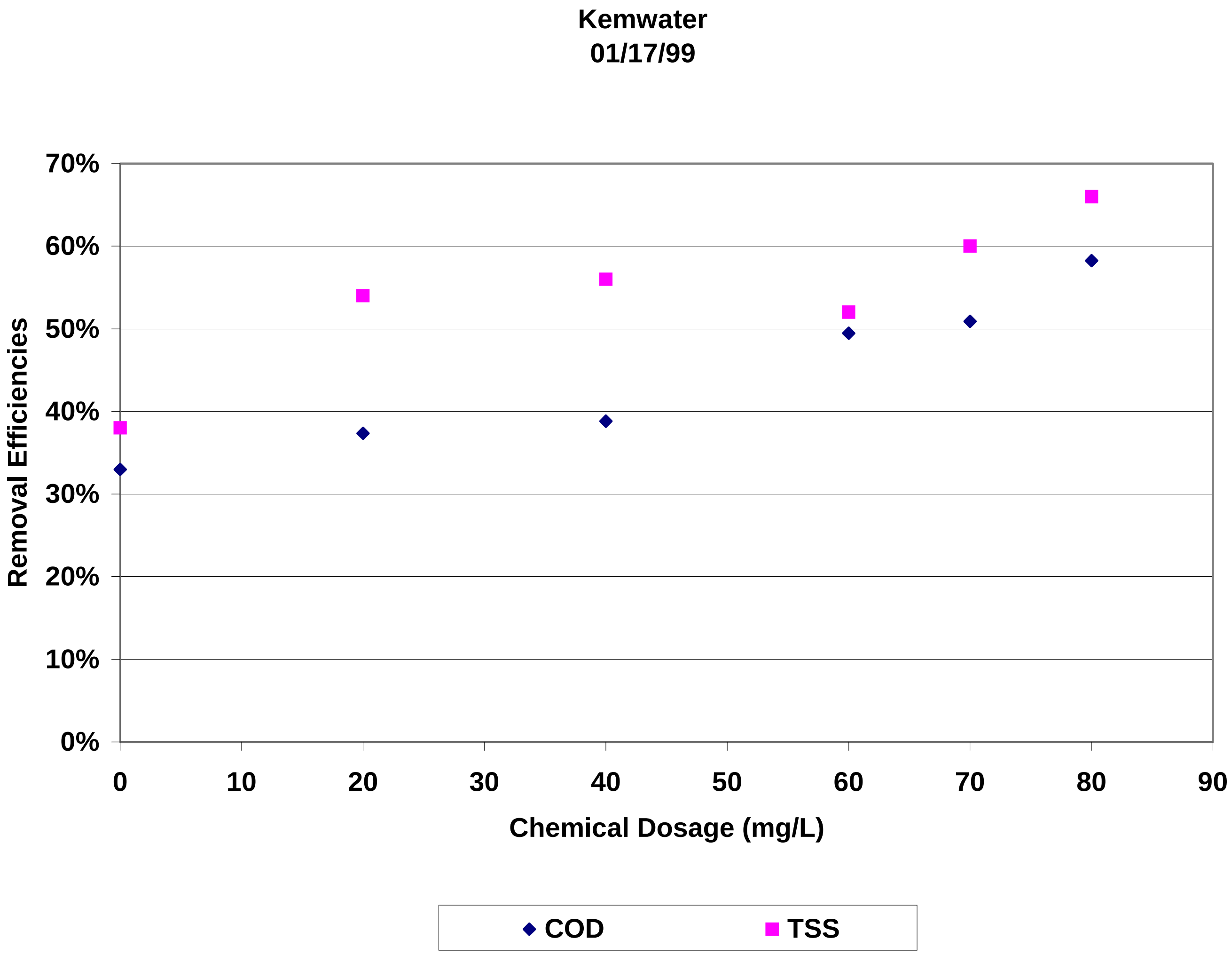


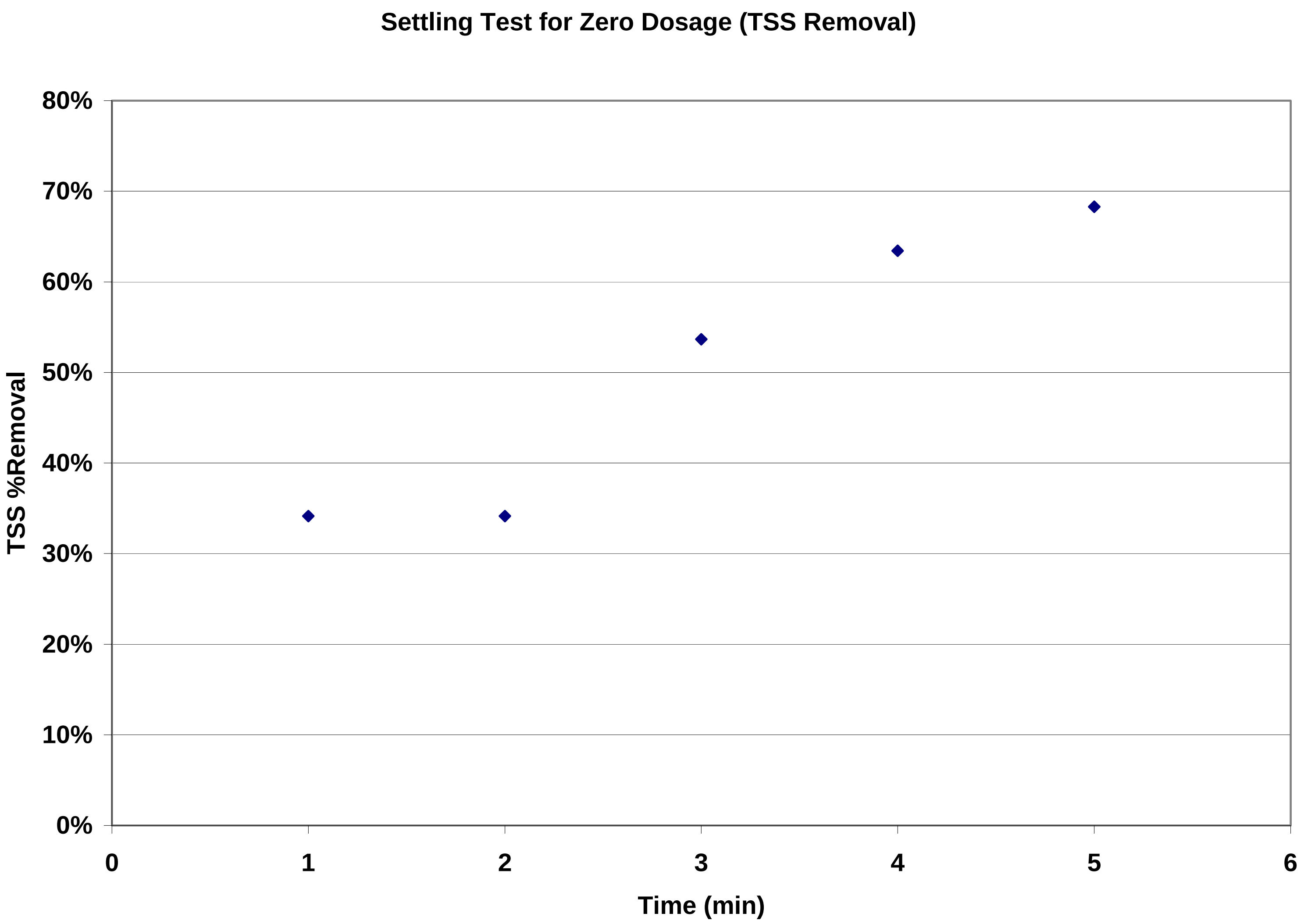

Liex
01/17/99
Removal Efficiencies
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
10
20
30
40
50
60
Chemical Dosage (mg/L)
◆ TSS
■ COD

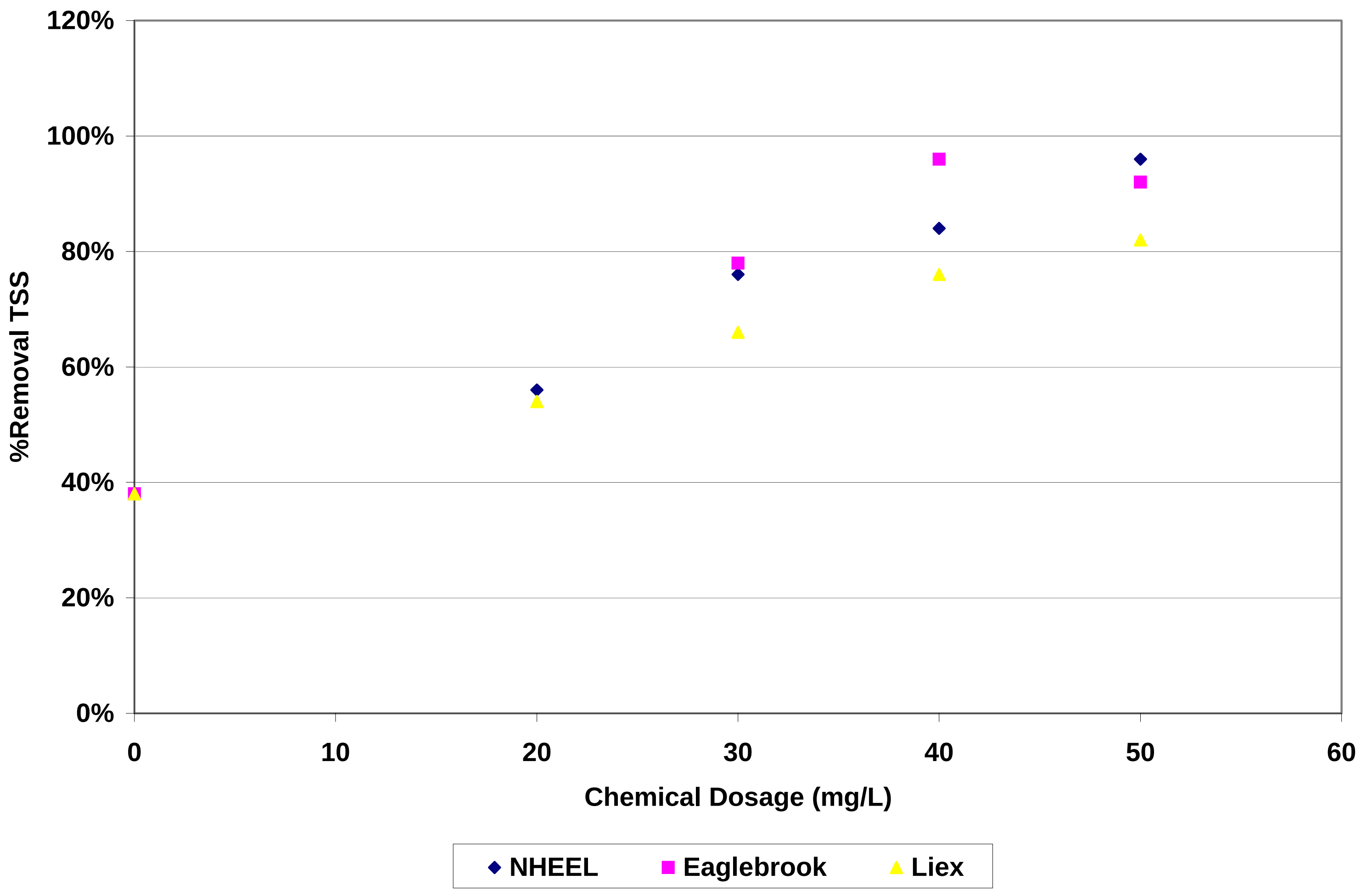

Eaglebrook Settling Test (TSS Removal)
Dosage = 50 mg/L
TSS %Removal
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
1
2
3
4
5
6
Time (min)


Eaglebrook Settling Test (COD Removal)
Dosage = 50 mg/L
COD %Removal
60%
50%
40%
30%
20%
10%
0%
0
1
2
3
4
5
6
Time (min)

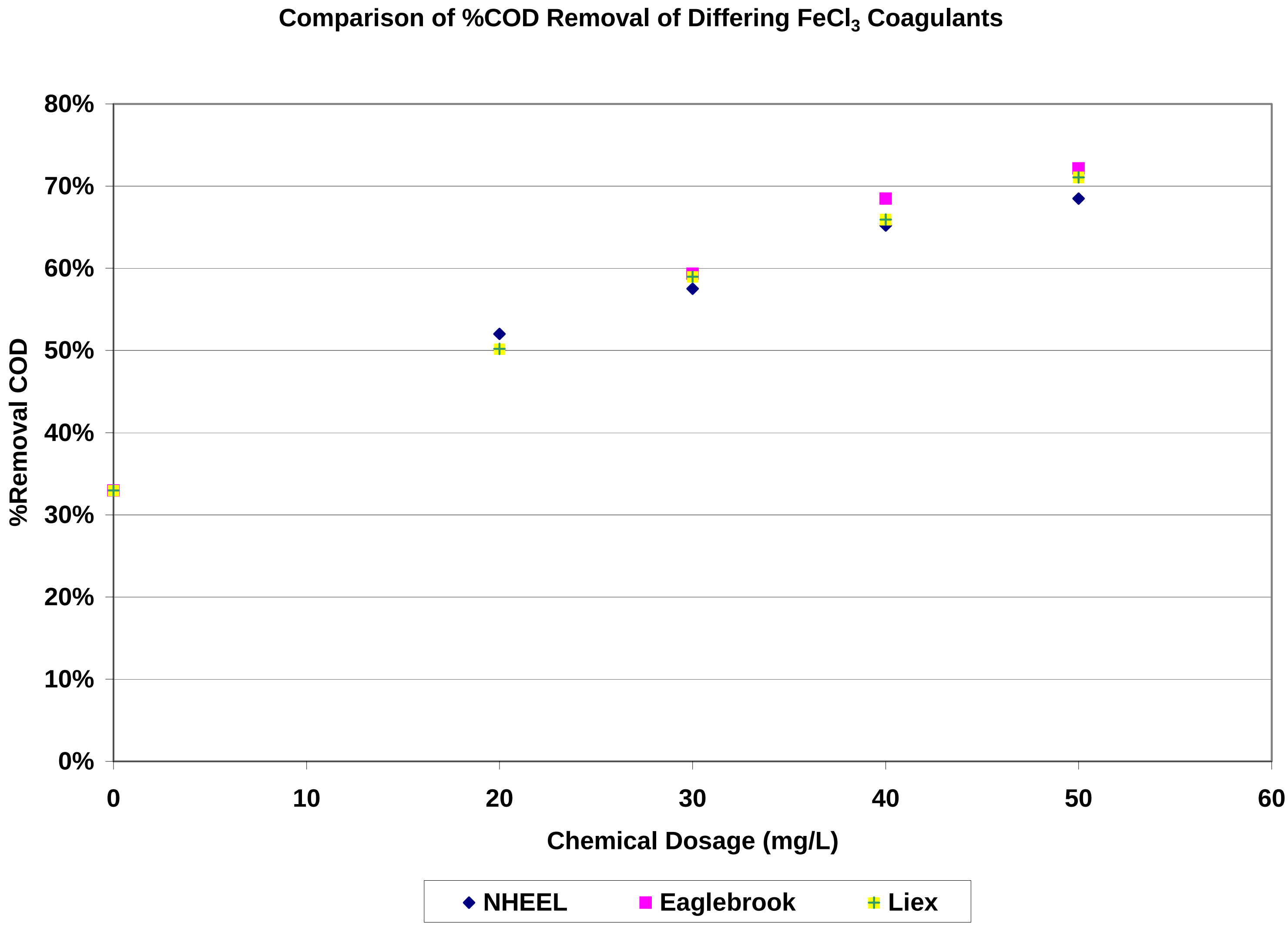

Eaglebrook
01/17/99
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Removal Efficiencies
0
10
20
30
40
50
60
Chemical Dosage (mg/L)
◆ COD
■ TSS


Kemwater
01/17/99
70%
60%
50%
40%
30%
20%
10%
0%
Removal Efficiencies
0
10
20
30
40
50
60
70
80
90
Chemical Dosage (mg/L)
◆ COD
■ TSS


Settling Test for Zero Dosage (TSS Removal)
TSS %Removal
80%
70%
60%
50%
40%
30%
20%
10%
0%
0
1
2
3
4
5
6
Time (min)



Comparison of %TSS Removal of Differing $FeCl_3$ Coagulants


Comparison of %COD Removal of Differing $FeCl_3$ Coagulants
80%
70%
60%
50%
40%
30%
20%
10%
0%
%Removal COD
0
10
20
30
40
50
60
Chemical Dosage (mg/L)
◆ NHEEL
■ Eaglebrook
+ Liex

ID#	Date	Time	Sampling Location	Sampling Time	Sample (Jar #)	Coagulant
1	15-Jan	15:00	splitter box	11:00	1	Eaglebrook
2	15-Jan	15:00	splitter box	11:00	2	Eaglebrook
3	15-Jan	15:00	splitter box	11:00	3	Eaglebrook
4	15-Jan	15:00	splitter box	11:00	4	Eaglebrook
5	15-Jan	15:00	splitter box	11:00	Raw	None
6	15-Jan	15:30	splitter box	11:00	1	Eaglebrook
7	15-Jan	15:30	splitter box	11:00	2	Eaglebrook
8	15-Jan	15:30	splitter box	11:00	3	Eaglebrook
9	15-Jan	15:30	splitter box	11:00	4	Eaglebrook
10	15-Jan	16:00	splitter box	11:00	1	Zero
11	15-Jan	16:00	splitter box	11:00	2	Liex
12	15-Jan	16:00	splitter box	11:00	3	Liex
13	15-Jan	16:00	splitter box	11:00	4	Liex
14	15-Jan	17:00	splitter box	11:00	1	Liex
15	15-Jan	17:00	splitter box	11:00	2	Liex
16	15-Jan	17:00	splitter box	11:00	3	Liex
17	15-Jan	17:00	splitter box	11:00	4	Liex
18	15-Jan	17:30	splitter box	11:00	1	Kemwater
19	15-Jan	17:30	splitter box	11:00	2	Kemwater
20	15-Jan	17:30	splitter box	11:00	3	Kemwater
21	15-Jan	17:30	splitter box	11:00	4	Kemwater
22	15-Jan	18:00	splitter box	11:00	1	Kemwater
23	15-Jan	18:00	splitter box	11:00	2	Kemwater
24	15-Jan	18:00	splitter box	11:00	3	Kemwater
25	16-Jan	13:00	splitter box	10:00	1	Sanechlor
26	16-Jan	13:00	splitter box	10:00	2	Sanechlor
27	16-Jan	13:00	splitter box	10:00	3	Sanechlor
28	16-Jan	13:00	splitter box	10:00	4	Sanechlor
29	16-Jan	13:00	B1	10:00	B1	None
30	16-Jan	13:00	J1	10:00	J1	None
31	16-Jan	13:30	splitter box	10:00	1	Zero
32	16-Jan	13:30	splitter box	10:00	2	NHEEL
33	16-Jan	13:30	splitter box	10:00	3	NHEEL
34	16-Jan	13:30	splitter box	10:00	4	NHEEL
35	16-Jan	14:00	splitter box	10:00	1	NHEEL
36	16-Jan	14:00	splitter box	10:00	2	Eaglebrook
37	16-Jan	14:00	splitter box	10:00	3	Sanechlor
38	16-Jan	14:00	splitter box	10:00	4	Kemwater
39	16-Jan	14:00	splitter box	10:00	5	Liex
40	16-Jan	16:45	splitter box	10:00	1	Sludge
41	16-Jan	16:45	splitter box	10:00	2	Sludge
42	16-Jan	16:45	splitter box	10:00	3	Sludge
43	16-Jan	16:45	splitter box	10:00	4	Sludge
44	16-Jan	16:45	B1	10:00	B1	None
45	16-Jan	16:45	J1	10:00	J1	None
46	16-Jan	17:15	splitter box	10:00	1	NHEEL + sl.
47	16-Jan	17:15	splitter box	10:00	2	NHEEL + sl.
48	16-Jan	17:15	splitter box	10:00	3	NHEEL + sl.
49	16-Jan	17:15	splitter box	10:00	4	NHEEL + sl.
50	16-Jan	17:15	splitter box	18:00	Raw @18:00	None

51	16-Jan	17:15	J1	10:00	J1	None
52	16-Jan	17:15	splitter box	10:00	Raw	None
53	17-Jan	12:45	splitter box	10:20	Raw	None
54	17-Jan	12:45	splitter box	10:20	1	Liex
55	17-Jan	12:45	splitter box	10:20	2	Liex
56	17-Jan	12:45	splitter box	10:20	3	Liex
57	17-Jan	12:45	splitter box	10:20	4	Liex
58	17-Jan	15:15	splitter box	10:20	1	Zero
59	17-Jan	15:15	splitter box	10:20	2	Eaglebrook
60	17-Jan	15:15	splitter box	10:20	3	Eaglebrook
61	17-Jan	15:15	splitter box	10:20	4	Eaglebrook
62	17-Jan	15:45	splitter box	10:20	1	NHEEL
63	17-Jan	15:45	splitter box	10:20	2	NHEEL
64	17-Jan	15:45	splitter box	10:20	3	NHEEL
65	17-Jan	15:45	splitter box	10:20	4	NHEEL
66	17-Jan	16:10	splitter box	10:20	1	Kemwater
67	17-Jan	16:10	splitter box	10:20	2	Kemwater
68	17-Jan	16:10	splitter box	10:20	3	Kemwater
69	17-Jan	16:10	splitter box	10:20	4	Kemwater
70	17-Jan	17:00	splitter box	10:20	1	Sanechlor
71	17-Jan	17:00	splitter box	10:20	2	Sanechlor
72	17-Jan	17:00	splitter box	10:20	3	Sanechlor
73	17-Jan	17:00	splitter box	10:20	4	Sanechlor
74	17-Jan	17:30	splitter box	10:20	1	Liex
75	17-Jan	17:30	splitter box	10:20	2	Sanechlor
76	17-Jan	17:30	splitter box	10:20	3	NHEEL
77	17-Jan	17:30	splitter box	10:20	4	Kemwater
78	17-Jan	18:00	splitter box	10:20	1	Liex
79	17-Jan	18:00	splitter box	10:20	2	Sanechlor
80	17-Jan	18:00	splitter box	10:20	3	NHEEL
81	17-Jan	18:00	splitter box	10:20	4	Kemwater
82	17-Jan	18:00	splitter box	10:20	1	Zero
83	17-Jan	18:00	splitter box	10:20	Raw-brl btm	None
84	17-Jan	NA	B1	10:20	B1	None
85	17-Jan	NA	J1	10:20	J1	None
86	17-Jan	NA	splitter box	15:00	Raw	None
87	17-Jan	NA	G1	15:00	NA	None
88	17-Jan	NA	J1	15:00	NA	None
89	17-Jan	NA	splitter box	10:20	NA	None
90	18-Jan	11:30	splitter box	9:45	Raw	None
91	18-Jan	11:30	splitter box	9:45	1	GAC Alum
92	18-Jan	11:30	splitter box	9:45	2	GAC Alum
93	18-Jan	11:30	splitter box	9:45	3	GAC Alum
94	18-Jan	11:30	splitter box	9:45	4	GAC Alum
95	18-Jan	13:15	splitter box	9:45	1 min	Liex
96	18-Jan	13:15	splitter box	9:45	2 min	Liex
97	18-Jan	13:15	splitter box	9:45	3 min	Liex
98	18-Jan	13:15	splitter box	9:45	4 min	Liex
99	18-Jan	13:15	splitter box	9:45	5 min	Liex
100	18-Jan	13:45	splitter box	9:45	1 min	NHEEL

101	18-Jan	13:45	splitter box	9:45	2 min	NHEEL
102	18-Jan	13:45	splitter box	9:45	3 min	NHEEL
103	18-Jan	13:45	splitter box	9:45	4 min	NHEEL
104	18-Jan	13:45	splitter box	9:45	5 min	NHEEL
105	18-Jan	14:00	splitter box	9:45	1 min	Eaglebrook
106	18-Jan	14:00	splitter box	9:45	2 min	Eaglebrook
107	18-Jan	14:00	splitter box	9:45	3 min	Eaglebrook
108	18-Jan	14:00	splitter box	9:45	4 min	Eaglebrook
109	18-Jan	14:00	splitter box	9:45	5 min	Eaglebrook
110	18-Jan	15:00	splitter box	9:45	1 min	Sanechlor
111	18-Jan	15:00	splitter box	9:45	2 min	Sanechlor
112	18-Jan	15:00	splitter box	9:45	3 min	Sanechlor
113	18-Jan	15:00	splitter box	9:45	4 min	Sanechlor
114	18-Jan	15:00	splitter box	9:45	5 min	Sanechlor
115	18-Jan	15:00	splitter box	9:45	Raw	None
116	18-Jan	15:00	G1	9:45	G1	None
117	18-Jan	15:00	J1	9:45	J1	None
118	18-Jan	16:30	splitter box	9:45	1 min	None
119	18-Jan	16:30	splitter box	9:45	2 min	None
120	18-Jan	16:30	splitter box	9:45	3 min	None
121	18-Jan	16:30	splitter box	9:45	4 min	None
122	18-Jan	16:30	splitter box	9:45	5 min	None
123	18-Jan	16:30	splitter box	9:45	1	Kemwater (new)
124	18-Jan	16:30	splitter box	9:45	2	Kemwater (old)
125	18-Jan	16:30	splitter box	15:30	Raw	None
126	18-Jan	16:30	G1	15:30	G1	None
127	18-Jan	16:30	J1	15:30	J1	None
128	19-Jan	12:00	splitter box	10:00	1	Zero
129	19-Jan	12:00	splitter box	10:00	2	Alum
130	19-Jan	12:00	splitter box	10:00	3	Alum
131	19-Jan	12:00	splitter box	10:00	4	Alum
132	19-Jan	12:30	splitter box	10:00	1	NHEEL
133	19-Jan	12:30	splitter box	10:00	2	NHEEL
134	19-Jan	12:30	splitter box	10:00	3	NHEEL
135	19-Jan	12:30	splitter box	10:00	4	NHEEL
136	19-Jan	13:00	splitter box	10:00	1	Eaglebrook
137	19-Jan	13:00	splitter box	10:00	2	Eaglebrook
138	19-Jan	13:00	splitter box	10:00	3	Eaglebrook
139	19-Jan	13:00	splitter box	10:00	4	Eaglebrook
140	19-Jan	13:30	splitter box	10:00	1	Kemwater
141	19-Jan	13:30	splitter box	10:00	2	Kemwater
142	19-Jan	13:30	splitter box	10:00	3	Kemwater
143	19-Jan	13:30	splitter box	10:00	4	Kemwater
144	19-Jan	14:10	splitter box	10:00	1	Liex
145	19-Jan	14:10	splitter box	10:00	2	Liex
146	19-Jan	14:10	splitter box	10:00	3	Sanechlor
147	19-Jan	14:10	splitter box	10:00	4	Sanechlor
148	19-Jan	17:00	splitter box	10:00	1	NHEEL
149	19-Jan	17:00	splitter box	10:00	2	NHEEL
150	19-Jan	17:00	splitter box	10:00	3	NHEEL

151	19-Jan	17:00	splitter box	10:00	4	Liex
152	19-Jan	18:00	splitter box	10:00	1	NHEEL
153	19-Jan	18:00	splitter box	10:00	2	NHEEL
154	19-Jan	18:00	splitter box	10:00	3	Liex
155	19-Jan	18:00	splitter box	10:00	4	Eaglebrook
156	19-Jan	18:30	splitter box	10:00	1	NHEEL
157	19-Jan	18:30	splitter box	10:00	2	NHEEL
158	19-Jan	18:30	splitter box	10:00	3	Liex
159	19-Jan	19:00	splitter box	10:00	1	Sanechlor
160	19-Jan	19:00	splitter box	10:00	2	NHEEL
161	19-Jan	19:00	splitter box	10:00	3	Liex
162	19-Jan	19:00	splitter box	10:00	4	Eaglebrook
163	20-Jan	15:20	splitter box	14:30	1	Sanechlor
164	20-Jan	15:20	splitter box	14:30	2	NHEEL
165	20-Jan	15:20	splitter box	14:30	3	Sanechlor
166	20-Jan	15:20	splitter box	14:30	4	NHEEL
167	20-Jan	15:45	splitter box	14:30	1	Sanechlor
168	20-Jan	15:45	splitter box	14:30	2	NHEEL
169	20-Jan	15:45	splitter box	14:30	3	Sanechlor
170	20-Jan	15:45	splitter box	14:30	4	NHEEL
171	20-Jan	16:20	splitter box	14:30	1	Sanechlor
172	20-Jan	16:20	splitter box	14:30	2	NHEEL
173	20-Jan	17:15	splitter box	14:30	1	NHEEL
174	20-Jan	17:15	splitter box	14:30	2	NHEEL
175	20-Jan	17:15	splitter box	14:30	3	NHEEL
176	20-Jan	18:20	splitter box	14:30	1	NHEEL
177	20-Jan	18:20	splitter box	14:30	2	Alum (1.342)
178	21-Jan	11:00	splitter box	10:45	Raw	None
179	21-Jan	11:00	splitter box	10:45	2	Sanechlor
180	21-Jan	11:00	splitter box	10:45	3	Sanechlor
181	21-Jan	11:00	splitter box	10:45	4	Sanechlor
182	21-Jan	12:15	splitter box	10:45	1	Sanechlor
183	21-Jan	12:15	splitter box	10:45	2	Sanechlor
184	21-Jan	12:15	splitter box	10:45	3	Zero
185	21-Jan	13:00	splitter box	10:45	1	Sanechlor + Sl
186	21-Jan	13:00	splitter box	10:45	2	Sanechlor + Sl
187	21-Jan	13:00	splitter box	10:45	3	Sanechlor + Sl

ID#	Chemical Dosage (mg/L)	Polymer	Polymer Dosage (mg/l)	COD (mg/L)	COD %removal	TSS (mg/L)
1	0	None	0	237	24%	46
2	10	None	0	248	21%	44
3	20	None	0	247	21%	44
4	20	None	0	194	38%	50
5	0	None	0	312	NA	104
6	40	None	0	125	60%	6
7	50	None	0	121	61%	0
8	60	None	0	121	61%	12
9	70	None	0	122	61%	14
10	0	None	0	240	23%	34
11	10	None	0	235	25%	46
12	20	None	0	225	28%	46
13	30	None	0	199	36%	50
14	40	None	0	135	57%	16
15	50	None	0	119	62%	-4
16	60	None	0	121	61%	0
17	70	None	0	117	63%	10
18	10	None	0	239	23%	44
19	20	None	0	228	27%	38
20	30	None	0	225	28%	40
21	40	None	0	210	33%	46
22	50	None	0	205	34%	48
23	60	None	0	188	40%	54
24	70	None	0	178	43%	52
25	20	None	0	171	17%	50
26	30	None	0	142	31%	50
27	40	None	0	128	38%	50
28	60	None	0	96	53%	8
29	0	None	0	135	57%	94
30	0	None	0	77	75%	44
31	0	None	0	162	21%	52
32	30	None	0	112	45%	NA
33	40	None	0	88	57%	6
34	60	None	0	72	65%	12
35	50	None	0	68	67%	NA
36	50	None	0	76	63%	2
37	50	None	0	94	54%	24
38	50	None	0	151	26%	50
39	50	None	0	85	59%	14
40	10	None	0	166	19%	30
41	30	None	0	170	17%	36
42	50	None	0	152	26%	30
43	60	None	0	149	27%	24
44	0	None	0	153	51%	94
45	0	None	0	91	71%	52
46	10	None	0	151	26%	40
47	20	None	0	120	41%	32
48	30	None	0	103	50%	26
49	40	None	0	99	52%	18
50	0	None	0	327	NA	130

51	0	None	0	98	69%	144
52	0	None	0	205	NA	87
53	0	None	0	183	NA	62
54	20	None	0	136	50%	46
55	30	None	0	112	59%	34
56	40	None	0	93	66%	24
57	50	None	0	79	71%	18
58	0	None	0	183	33%	62
59	30	None	0	111	59%	22
60	40	None	0	86	68%	4
61	50	None	0	76	72%	8
62	20	None	0	131	52%	44
63	30	None	0	116	58%	24
64	40	None	0	95	65%	16
65	50	None	0	86	68%	4
66	20	None	0	171	37%	46
67	40	None	0	167	39%	44
68	60	None	0	138	49%	48
69	80	None	0	114	58%	34
70	20	None	0	162	41%	52
71	40	None	0	126	54%	36
72	60	None	0	91	67%	6
73	80	None	0	82	70%	6
74	40	None	0	111	59%	14
75	70	None	0	91	67%	8
76	40	None	0	108	60%	14
77	70	None	0	134	51%	40
78	20	None	0	137	50%	30
79	35	None	0	NA	NA	NA
80	20	None	0	173	37%	32
81	35	None	0	NA	NA	NA
82	0	None	0	192	30%	38
83	0	None	0	366	NA	154
84	0	None	0	130	58%	80
85	0	None	0	88	72%	38
86	0	None	0	361	NA	68
87	0	None	0	182	42%	104
88	0	None	0	107	66%	50
89	0	None	0	273	NA	100
90	0	None	0	267	NA	164
91	20	None	0	NA	NA	NA
92	40	None	0	NA	NA	NA
93	60	None	0	NA	NA	NA
94	80	None	0	NA	NA	NA
95	50	None	0	268	0%	154
96	50	None	0	168	37%	56
97	50	None	0	138	48%	36
98	50	None	0	135	49%	36
99	50	None	0	134	50%	32
100	50	None	0	260	3%	148

101	50	None	0	152	43%	48
102	50	None	0	130	51%	32
103	50	None	0	133	50%	32
104	50	None	0	139	48%	28
105	50	None	0	231	13%	128
106	50	None	0	123	54%	16
107	50	None	0	137	49%	40
108	50	None	0	121	55%	20
109	50	None	0	123	54%	20
110	50	None	0	301	-13%	196
111	50	None	0	238	11%	112
112	50	None	0	197	26%	84
113	50	None	0	184	31%	72
114	50	None	0	182	32%	76
115	0	None	0	158	NA	108
116	0	None	0	195	37%	104
117	0	None	0	105	66%	44
118	0	None	0	291	-9%	108
119	0	None	0	320	-20%	108
120	0	None	0	283	-6%	76
121	0	None	0	273	-2%	60
122	0	None	0	261	2%	52
123	60	None	0	NA	NA	NA
124	60	None	0	NA	NA	NA
125	0	None	0	429	NA	152
126	0	None	0	238	24%	108
127	0	None	0	142	54%	64
128	0	None	0	338	NA	36
129	20	None	0	357	NA	40
130	40	None	0	299	NA	44
131	60	None	0	303	NA	44
132	30	#13	0.5	NA	NA	NA
133	30	#15	0.5	NA	NA	NA
134	30	#17	0.5	NA	NA	NA
135	30	#19	0.5	NA	NA	NA
136	30	#13	0.5	NA	NA	NA
137	30	#15	0.5	NA	NA	NA
138	30	#17	0.5	NA	NA	NA
139	30	#19	0.5	NA	NA	NA
140	30	#13	0.5	NA	NA	NA
141	30	#15	0.5	NA	NA	NA
142	30	#17	0.5	NA	NA	NA
143	30	#19	0.5	NA	NA	NA
144	30	#15	0.5	NA	NA	NA
145	30	#17	0.5	NA	NA	NA
146	30	#15	0.5	NA	NA	NA
147	30	#17	0.5	NA	NA	NA
148	30	#17	0.5	NA	NA	NA
149	30	S-non	5	NA	NA	NA
150	30	#17	0.5	NA	NA	NA

151	30	#17	0.5	NA	NA	NA
152	30	S-non	5	NA	NA	NA
153	30	#17	0.5	NA	NA	NA
154	30	#17	0.5	NA	NA	NA
155	30	#17	0.5	NA	NA	NA
156	34	S-non	0.26	NA	NA	NA
157	34	#17	0.26	NA	NA	NA
158	34	#17	0.26	NA	NA	NA
159	50	#17	0.3	NA	NA	NA
160	30	#17	0.3	NA	NA	NA
161	30	#17	0.3	NA	NA	NA
162	30	#17	0.3	NA	NA	NA
163	60	#17	0.3	311	NA	28
164	30	#17	0.2	327	NA	48
165	50	#17	0.2	291	NA	36
166	25	#17	0.15	312	NA	48
167	50	#17	0.25	273	NA	24
168	25	#17	0.25	323	NA	48
169	40	#17	0.3	306	NA	40
170	35	#17	0.15	293	NA	28
171	60	#17	0.15	256	NA	16
172	35	#17	0.25	329	NA	32
173	25	#17	0.5	NA	NA	NA
174	25	S-cat	0.5	NA	NA	NA
175	25	S-an	0.5	NA	NA	NA
176	40	#17	0.3	NA	NA	NA
177	40	#17	0.3	NA	NA	NA
178	0	None	0	565	NA	300
179	42	#17	0.25	297	47%	36
180	46	#17	0.25	NA	NA	24
181	50	#17	0.25	NA	NA	32
182	42	#17	0.15	274	52%	32
183	42	#17	0.2	NA	NA	20
184	0	None	0	374	34%	64
185	40 + 25	#17	0.1	307	46%	44
186	40 + 40	#17	0.1	304	46%	44
187	40 + 0	#17	0.1	288	49%	36

ID#	TSS %removal	SO4	PO4	Floc Size	Visual Observation	Sample Volume
1	55.77%	NA	NA	NA	*Jar 3 had black specks	1L
2	57.69%	NA	NA	NA	that settled to the bottom	1L
3	57.69%	NA	NA	NA	and showed up on the .	1L
4	51.92%	NA	NA	NA	TSS filter.	1L
5	0.00%	NA	NA	NA	NA	NA
6	94.23%	NA	NA	b	NA	1L
7	100.00%	NA	NA	B	NA	1L
8	88.46%	NA	NA	c	NA	1L
9	86.54%	NA	NA	C	NA	1L
10	67.31%	NA	NA	-	NA	1L
11	55.77%	NA	NA	a	NA	1L
12	55.77%	NA	NA	a	NA	1L
13	51.92%	NA	NA	a	NA	1L
14	84.62%	NA	NA	B	NA	1L
15	103.85%	NA	NA	B	NA	1L
16	100.00%	NA	NA	B	NA	1L
17	90.38%	NA	NA	c	NA	1L
18	57.69%	NA	NA	NA	NA	1L
19	63.46%	NA	NA	NA	NA	1L
20	61.54%	NA	NA	NA	NA	1L
21	55.77%	NA	NA	NA	NA	1L
22	53.85%	NA	NA	B	NA	1L
23	48.08%	NA	NA	B	NA	1L
24	50.00%	NA	NA	B	NA	1L
25	43%	NA	NA	A	cloudy-green	1L
26	43%	NA	NA	C	cloudy-green	1L
27	43%	NA	NA	d	less green, less cloudy	1L
28	91%	NA	NA	D	clear supernatant	1L
29	29%	NA	NA	NA	NA	NA
30	67%	NA	NA	NA	NA	NA
31	40%	NA	NA	-	very cloudy	1L
32	NA	NA	NA	C	cloudy	1L
33	93%	NA	NA	d	partially clear	1L
34	86%	NA	NA	D	very clear	1L
35	NA	NA	NA	e	clearest, pin floc	1L
36	98%	NA	NA	e	very clear, no pin floc	1L
37	72%	NA	NA	e	golden/cloudy	1L
38	43%	NA	NA	B	cloudy, worst	1L
39	84%	NA	NA	D	small amount of pin floc	1L
40	66%	NA	NA	b	NA	1L
41	59%	NA	NA	b	NA	1L
42	66%	NA	NA	b	NA	1L
43	72%	NA	NA	c	NA	1L
44	29%	NA	NA	NA	NA	NA
45	61%	NA	NA	NA	NA	NA
46	54%	NA	NA	c	least clear	1L
47	63%	NA	NA	C	NA	1L
48	70%	NA	NA	d	NA	1L
49	79%	NA	NA	D	clearest	1L
50	NA	NA	NA	NA	NA	NA

51	-8%	NA	NA	NA	NA	NA
52	NA	NA	NA	NA	NA	NA
53	NA	NA	NA	NA	NA	NA
54	54%	NA	NA	A	very cloudy, lots of pin floc	1L
55	66%	NA	NA	B	cloudy, some pin floc	1L
56	76%	NA	NA	c	hazy/golden	1L
57	82%	NA	NA	C	clear	1L
58	38%	NA	NA	-	NA	1L
59	78%	NA	NA	d	golden-some pin floc	1L
60	96%	NA	NA	D	slightly golden - little pin floc	1L
61	92%	NA	NA	E	clear	1L
62	56%	NA	NA	c	lots of pin floc	1L
63	76%	NA	NA	C	golden - some pin floc	1L
64	84%	NA	NA	D	clear	1L
65	96%	NA	NA	E	clear	1L
66	54%	NA	NA	A	cloudy	1L
67	56%	NA	NA	b	cloudy	1L
68	52%	NA	NA	c	cloudy	1L
69	66%	NA	NA	d	golden, pin floc	1L
70	48%	NA	NA	b	very cloudy	1L
71	64%	NA	NA	d	cloudy	1L
72	94%	NA	NA	e	cloudy	1L
73	94%	NA	NA	E	clear	1L
74	86%	NA	NA	e	NA	1L
75	92%	NA	NA	E	NA	1L
76	86%	NA	NA	e	NA	1L
77	60%	NA	NA	d	NA	1L
78	70%	NA	NA	e	NA	1L
79	NA	NA	NA	e	NA	1L
80	68%	NA	NA	e	NA	1L
81	NA	NA	NA	C	NA	1L
82	62%	NA	NA	NA	NA	1L
83	NA	NA	NA	NA	NA	NA
84	40%	NA	NA	NA	NA	NA
85	71%	NA	NA	NA	NA	NA
86	NA	NA	NA	NA	NA	NA
87	22%	NA	NA	NA	NA	NA
88	62%	NA	NA	NA	NA	NA
89	NA	NA	NA	NA	NA	NA
90	NA	42	>2.75	NA	NA	NA
91	NA	NA	NA	A	cloudy	1L
92	NA	NA	NA	c	cloudy	1L
93	NA	NA	NA	C	golden, lots of pin floc	1L
94	NA	NA	NA	D	lots of pin floc	1L
95	6%	NA	NA	NA	NA	2L
96	66%	NA	NA	NA	NA	2L
97	78%	NA	NA	NA	NA	2L
98	78%	NA	NA	NA	NA	2L
99	80%	NA	NA	NA	NA	2L
100	10%	NA	NA	NA	NA	2L

101	71%	NA	NA	NA	NA	2L
102	80%	NA	NA	NA	NA	2L
103	80%	NA	NA	NA	NA	2L
104	83%	NA	NA	NA	NA	2L
105	22%	NA	NA	NA	NA	2L
106	90%	NA	NA	NA	NA	2L
107	76%	NA	NA	NA	NA	2L
108	88%	NA	NA	NA	NA	2L
109	88%	NA	NA	NA	NA	2L
110	-20%	NA	NA	NA	NA	2L
111	32%	NA	NA	NA	NA	2L
112	49%	NA	NA	NA	NA	2L
113	56%	NA	NA	NA	NA	2L
114	54%	NA	NA	NA	NA	2L
115	NA	NA	NA	NA	NA	NA
116	22%	NA	NA	NA	NA	NA
117	67%	NA	NA	NA	NA	NA
118	34%	NA	NA	NA	NA	1L
119	34%	NA	NA	NA	NA	1L
120	54%	NA	NA	NA	NA	1L
121	63%	NA	NA	NA	NA	1L
122	68%	NA	NA	NA	NA	1L
123	NA	NA	NA	C	cloudy	1L
124	NA	NA	NA	C	cloudy	1L
125	NA	NA	NA	NA	NA	NA
126	19%	NA	NA	NA	NA	NA
127	52%	NA	NA	NA	NA	NA
128	NA	NA	NA	-	*All were very cloudy with	1L
129	NA	NA	NA	B	lots of pin floc.	1L
130	NA	NA	NA	B		1L
131	NA	NA	NA	c		1L
132	NA	NA	NA	F	worst	1L
133	NA	NA	NA	>>G	best	1L
134	NA	NA	NA	>>G	a close second	1L
135	NA	NA	NA	>>G	third	1L
136	NA	NA	NA	D	last	1L
137	NA	NA	NA	>>G	close second	1L
138	NA	NA	NA	>>G	best	1L
139	NA	NA	NA	>>G	third	1L
140	NA	NA	NA	D	4th	1L
141	NA	NA	NA	>G	1st	1L
142	NA	NA	NA	>G	close 2nd	1L
143	NA	NA	NA	F	3rd	1L
144	NA	NA	NA	>>G	2nd	1L
145	NA	NA	NA	>>G	1st	1L
146	NA	NA	NA	>>G	4th	1L
147	NA	NA	NA	>>G	3rd	1L
148	NA	NA	NA	>>G	NA	1L
149	NA	NA	NA	>>G	NA	1L
150	NA	NA	NA	>>G	NA	1L

151	NA	NA	NA	>>G	NA	1L
152	NA	NA	NA	>>G	*All performed similarly	1L
153	NA	NA	NA	>>G	*All were a bit foggy	1L
154	NA	NA	NA	>>G		1L
155	NA	NA	NA	>>G		1L
156	NA	NA	NA	f	NA	1L
157	NA	NA	NA	G	NA	1L
158	NA	NA	NA	G	NA	1L
159	NA	NA	NA	NA	2nd	1L
160	NA	NA	NA	NA	3rd	1L
161	NA	NA	NA	NA	4th	1L
162	NA	NA	NA	NA	1st	1L
163	NA	NA	NA	NA	NA	1L
164	NA	NA	NA	NA	NA	1L
165	NA	NA	NA	NA	NA	1L
166	NA	NA	NA	NA	NA	1L
167	NA	NA	NA	F	3rd	1L
168	NA	NA	NA	>G	settled fasted	1L
169	NA	NA	NA	>G	2nd	1L
170	NA	NA	NA	d	4th	1L
171	NA	NA	NA	f	smaller flocs	1L
172	NA	NA	NA	G	settled much faster	1L
173	NA	NA	NA	>>G	*The #17 combo was by far	1L
174	NA	NA	NA	c	the best. The two provided	1L
175	NA	NA	NA	c	by SABESP were cloudy.	1L
176	NA	NA	NA	>>G	much better	1L
177	NA	NA	NA	F	not as good	1L
178	NA	75	8.12	NA	NA	NA
179	88%	66	5.88	>G	NA	2L
180	92%	NA	NA	G	NA	2L
181	89%	NA	NA	G	NA	2L
182	89%	65	5.52	F	settled fastest	2L
183	93%	NA	NA	F	NA	2L
184	79%	45	7.72	NA	NA	2L
185	85%	65	NA	F	NA	2L
186	85%	67	NA	F	NA	2L
187	88%	67	NA	F	NA	2L

ID#	**Purpose => Results**
1	Test Eaglebrook
2	Test Eaglebrook
3	Test Eaglebrook
4	Test Eaglebrook
5	NA
6	Test Eaglebrook
7	Test Eaglebrook
8	Test Eaglebrook
9	Test Eaglebrook
10	Test Liex
11	Test Liex
12	Test Liex
13	Test Liex
14	Test Liex
15	Test Liex
16	Test Liex
17	Test Liex
18	Test Kemwater
19	Test Kemwater
20	Test Kemwater
21	Test Kemwater
22	Test Kemwater
23	Test Kemwater
24	Test Kemwater
25	Test Sanechlor
26	Test Sanechlor
27	Test Sanechlor
28	Test Sanechlor
29	NA
30	NA
31	Test NHEEL
32	Test NHEEL
33	Test NHEEL
34	Test NHEEL
35	A side-by-side comparison of five Iron salts.
36	A side-by-side comparison of five Iron salts.
37	A side-by-side comparison of five Iron salts.
38	A side-by-side comparison of five Iron salts.
39	A side-by-side comparison of five Iron salts.
40	Test NHEEL-generated sludge as a coagulant w/o adding additional coagulants
41	Test NHEEL-generated sludge as a coagulant w/o adding additional coagulants
42	Test NHEEL-generated sludge as a coagulant w/o adding additional coagulants
43	Test NHEEL-generated sludge as a coagulant w/o adding additional coagulants
44	NA
45	NA
46	Test NHEEL at differing dosages with 30 ml of NHEEL-generated sludge.
47	Test NHEEL at differing dosages with 30 ml of NHEEL-generated sludge.
48	Test NHEEL at differing dosages with 30 ml of NHEEL-generated sludge.
49	Test NHEEL at differing dosages with 30 ml of NHEEL-generated sludge.
50	NA

51	NA
52	NA
53	NA
54	Liex series
55	Liex series
56	Liex series
57	Liex series
58	Eaglebrook series
59	Eaglebrook series
60	Eaglebrook series
61	Eaglebrook series
62	NHEEL series
63	NHEEL series
64	NHEEL series
65	NHEEL series
66	Kemwater Series
67	Kemwater Series
68	Kemwater Series
69	Kemwater Series
70	Sanechlor series
71	Sanechlor series
72	Sanechlor series
73	Sanechlor series
74	Side-by-side comparison
75	Side-by-side comparison
76	Side-by-side comparison
77	Side-by-side comparison
78	Side-by-side comparison
79	Side-by-side comparison
80	Side-by-side comparison
81	Side-by-side comparison
82	NA
83	NA
84	NA
85	NA
86	NA
87	NA
88	NA
89	NA
90	NA
91	GAC Alum series
92	GAC Alum series
93	GAC Alum series
94	GAC Alum series
95	Settling test for Liex
96	Settling test for Liex
97	Settling test for Liex
98	Settling test for Liex
99	Settling test for Liex
100	Settling test for NHEEL

101 Settling test for NHEEL
102 Settling test for NHEEL
103 Settling test for NHEEL
104 Settling test for NHEEL
105 Settling test for Eaglebrook
106 Settling test for Eaglebrook
107 Settling test for Eaglebrook
108 Settling test for Eaglebrook
109 Settling test for Eaglebrook
110 Settling test for Sanechlor
111 Settling test for Sanechlor
112 Settling test for Sanechlor
113 Settling test for Sanechlor
114 Settling test for Sanechlor
115 NA
116 NA
117 NA
118 Zero chemical settling test.
119 Zero chemical settling test.
120 Zero chemical settling test.
121 Zero chemical settling test.
122 Zero chemical settling test.
123 Compare old Kemwater to new Kemwater provided by SABESP.
124 Compare old Kemwater to new Kemwater provided by SABESP.
125 NA
126 NA
127 NA
128 Test Alum provided by SABESP => Very poor performance.
129 Test Alum provided by SABESP => Very poor performance.
130 Test Alum provided by SABESP => Very poor performance.
131 Test Alum provided by SABESP => Very poor performance.
132 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
133 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
134 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
135 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
136 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
137 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
138 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
139 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
140 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
141 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
142 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
143 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
144 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
145 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
146 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
147 To choose the best polymer (all are anionic)=> Polymer #17 performed best.
148 *To compare anionic polymer #17 to SABESP's non-ionic (S-non) polymer & compare NHEEL
149 to Liex (both using anionic polymer #17) to find out which will perform better with this specific
150 polymer. => Anionic polymer #17 performed much better than "S-non"; NHEEL was performed

151 better than Liex with anionic polymer #17.
152 *To compare the three best performing Iron (Fe) salts with anionic polymer #17 & to compare
153 anionic polymer #17 to SABESP's non-ionic (S-non). All Fe salts (with polymer #17) performed
154 very similarly, especially in the fact that they all formed very large floc (>>G) that settled very
155 rapidly (in a matter of seconds). S-non had nowhere near as good a performance.
156 Compare the performance of the polymers at lower dosages =>Nonionic poor, NHEEL better than Liex.
157 Compare the performance of the polymers at lower dosages =>Nonionic poor, NHEEL better than Liex.
158 Compare the performance of the polymers at lower dosages =>Nonionic poor, NHEEL better than Liex.
159 A side-by-side test to find the best performer at this dosage =>Eaglebrook was best, Sanechlor 2nd.
160 A side-by-side test to find the best performer at this dosage =>Eaglebrook was best, Sanechlor 2nd.
161 A side-by-side test to find the best performer at this dosage =>Eaglebrook was best, Sanechlor 2nd.
162 A side-by-side test to find the best performer at this dosage =>Eaglebrook was best, Sanechlor 2nd.
163 Compare Sanechlor with NHEEL at varying dosages to find the best Fe-Poly comination.
164 Compare Sanechlor with NHEEL at varying dosages to find the best Fe-Poly comination.
165 Compare Sanechlor with NHEEL at varying dosages to find the best Fe-Poly comination.
166 Compare Sanechlor with NHEEL at varying dosages to find the best Fe-Poly comination.
167 To compare NHEEL with Sanechlor.
168 To compare NHEEL with Sanechlor.
169 To compare NHEEL with Sanechlor.
170 To compare NHEEL with Sanechlor.
171 To compare Sanechlor and NHEEL.
172 To compare Sanechlor and NHEEL.
173 To compare anionic polymer #17 with SABESP's cationic (S-cat) and anionic (S-an) polymer.
174 To compare anionic polymer #17 with SABESP's cationic (S-cat) and anionic (S-an) polymer.
175 To compare anionic polymer #17 with SABESP's cationic (S-cat) and anionic (S-an) polymer.
176 To compare Aum (1.342) from SABESP to NHEEL.
177 To compare Aum (1.342) from SABESP to NHEEL.
178 NA
179 To find the best Sanechlor and polymer combination
180 To find the best Sanechlor and polymer combination
181 To find the best Sanechlor and polymer combination
182 To find the best Sanechlor and polymer combination
183 To find the best Sanechlor and polymer combination
184 To find the best Sanechlor and polymer combination
185 Test the recycling of chemical sludge at varying dosages.
186 Test the recycling of chemical sludge at varying dosages.
187 Test the recycling of chemical sludge at varying dosages.

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